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OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

INSTITUTED 1852

VOL. LXXXII

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ERRATA

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Page 1564, "Columbus, Ohio", should not have been listed under "Slow Sand Filter Plants", but under "Rapid Sand Filter Plants". The cost given is exclusive of land, but includes both a high- and a low-lift pumping station and a cast-iron force main. The reference to *Transactions*, Vol. XLVII, p. 261, should be Vol. LXVII, p. 260.

Page 1674, line 3, insert the words, "and Priscilla Mullins," after "Plymouth, Mass."

Page 1674, line 13, omit the words, "and, in 1878, became a member of the firm."

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Page 160, line 12, for "W. S. Darling", read "W. L. Darling."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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Paper No. 1403

THE CAPE COD CANAL*

BY WILLIAM BARCLAY PARSONS, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. CLEMENS HERSCHEL, T. KENNARD THOMSON, AND WILLIAM BARCLAY PARSONS.

SYNOPSIS.

The Cape Cod Canal joins Cape Cod Bay with the waters adjacent to Long Island Sound, traversing the narrow isthmus of Cape Cod. It has a length of 8 miles, with dredged approach channels 5 miles long. The minimum width of the canal is 100 ft., the maximum, 300 ft., and the depth at low water, 25 ft.

This canal was suggested for commercial and naval use 300 years ago. The project, originally a colonial one, subsequently became a national one, and finally was carried out by private capital. It has been in successful operation since the summer of 1914, and is now used by commercial and naval vessels.

Part I of this paper is devoted to history and location, and contains considerable data on construction. Part II is devoted entirely to hydraulics. The canal is the largest open artificial waterway connecting two seas having non-synchronous tides.

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* Presented at the meeting of October 3d, 1917.

PART I.

The southeast portion of the State of Massachusetts is a curiously shaped, narrow, hooked point, enclosing nearly three-quarters of a circle having a radius of about 12 miles. The south side of the enclosing land is extended southwestward to another point at Woods Hole, which, with a succession of islands, makes the barrier separating Buzzards Bay from Vineyard Sound. To this whole area of land the name of Cape Cod is applied, with Provincetown at the north, where the Pilgrims in the *Mayflower* made their first landing in 1621. The material composing the peninsula is sand, gravel, and granite boulders—an old glacial terminal moraine. The sand has been acted on by the currents of the Gulf Stream flowing east and the eddy of the Arctic Current sweeping down the New England coast, and this accounts for the peculiar contour of the shore. To the south and southeast of Cape Cod are various irregular shoals, known as Nantucket Shoals, the outer limit of which, in 30 fathoms of water, is marked by the Nantucket Light Vessel, distant from Monomoy Point, the extreme southernmost part of Cape Cod, 59 geographical or 68 statute miles in an almost due south direction.

These shoals of shifting sand, over which the depth varies from less than 1 fathom to 25 fathoms, discovered first by Verrazzano in 1524, and named by De Mont in 1605 "Mallebarre", have always been a terror to navigators. The extent, depth of water, and character of the Nantucket Shoals have been described in a report by Lt.-Col. (now Col.) J. C. Sanford, Corps of Engineers, U. S. A., to the Chief of Engineers, under date of November 16th, 1909, from which the following extracts have been taken:

"The numerous and extensive shoals lying eastward and southeastward of the eastern entrance to Nantucket Sound and southward and eastward of the southeasterly elbow of Cape Cod, constitute probably the greatest danger to navigation to be found on any of the coastwise routes of the Atlantic coast of the United States north of Hatteras. In view of the numerous vessels passing around these shoals they are probably a greater menace to navigation than Hatteras. Their dangerous character is shown both by the large number of wrecks annually occurring there and by the large number of light vessels and other aids to navigators traversing the shoals.

"From Nantucket Sound to the ocean two channels lead through the shoals. The north or Pollock Rip Channel is the most used, as it is shorter, is somewhat protected from easterly storms by the

shoals outside it, and is closer to the shore; but it is quite circuitous and narrow in places and the tidal currents are strong and varying in direction. The second or south channel leads through the shoals in a nearly due east direction from Nantucket (Great Point) Lighthouse. It is somewhat deeper than the Pollock Rip Channel and much wider, but it is not so direct for coastwise vessels and carries a vessel much farther from the shore. This channel is considered in the United States Coast Pilot for the Atlantic coast as the dividing line between Nantucket and Monomoy Shoals, the shoals lying to the northward of the channel being called the Monomoy Shoals, while those to the southward are called the Nantucket Shoals. The following description of the Monomoy Shoals in general and of the shoals particularly named in the river and harbor act, with others lying along the course of the proposed improvement, is taken from the above publication:

"'Monomoy Shoals consist of numerous detached shoals of a shifting character with 3 to 18 feet over them, extending about 5½ miles in an easterly and 9½ miles in a southerly and south-southeasterly direction from Monomoy Point. Many parts of these shoals separated from others by narrow slues have special names and are briefly described below:

"'Bearse Shoal is the western and Pollock Rip the eastern part of the shoal extending from ⅜ mile to 3¾ miles eastward of Monomoy Lighthouse. These shoals consist of a series of sand shoals and sand ridges, with 4 to 18 feet over them and deep water between them.

"'Broken Part of Pollock Rip, with depths of 15 to 18 feet over it, lies eastward of Pollock Rip, and is separated from it by Pollock Rip Slue, which has a width of about ½ mile and depth of 3½ to 6 fathoms.

"'Twelve Foot Shoal, southward of the broken part of Pollock Rip, has 14 to 18 feet over it and lies 5½ miles SE ½ E from Monomoy Lighthouse.

"'Stone Horse Shoal, Little Round Shoal, and Great Round Shoal are portions of a continuous series of sand shoals and sand ridges with depths of 5 to 18 feet over them, lying directly eastward of the entrance of Nantucket Sound and between the two main channels. Stone Horse Shoal and Little Round Shoal lie on the south side of the deepwater channel between them and Pollock Rip. Great Round Shoal lies from 6 to 9½ miles in SSE direction from Monomoy Point Lighthouse; southward and eastward of this shoal for a distance of about 2½ miles there are numerous shoal spots with depths varying from 17 to 18 feet over them.

"'Shovelful Shoal, extending ¾ mile southward from Monomoy Point, is bare in places and rises abruptly from the deep waters of Butlers Hole.

"'Handkerchief Shoal is the extensive shoal, with from 3 to 18 feet over it, lying southwestward of Monomoy Point. It is about 4¼

miles long north and south, and its greatest width is about 2 miles. Its southern end, which rises abruptly from a depth of 8 fathoms to 10 feet, is about ½ mile northward of Handkerchief Shoal Light vessel and 5¼ miles SW ¾ W from Monomoy Point Lighthouse. Its northern end rising gradually from 3½ fathoms to 15 feet, lies about 3 miles WNW ½ W from Monomoy Point Lighthouse.'

"The shoals are undoubtedly of a shifting character. A comparison of Coast Survey charts issued from 1860 to the present time shows enormous changes in the channels and in the shape and position of the various shoals.

"On the chart of 1860 the principal passage from Butlers Hole (deep water southwest of Shovelful Shoal Light Vessel) to the ocean was due east from Pollock Rip Light Vessel through a 5-fathom passage south of the broken part of Pollock Rip. The chart of 1874 shows this passage closed by the 5-fathom contour, which is continuous from off Chatham around the entire group of the Monomoy Shoals, the distance between the outside and inside 5-fathom curves being but 600 yards, with a depth of 4¾ fathoms between. The 1885 chart shows this distance to be about 800 yards, with 3¼ fathoms between and with several small shoals carrying less than 3 fathoms in the immediate vicinity. The 1888 chart shows this distance to be about 2 500 yards, with a minimum depth of 3¼ fathoms. The 1894 and 1900 charts give the distance as about 900 yards, with a minimum depth of 3¼ fathoms. The 1908 chart gives the extreme distance between the inside and outside 5-fathom contours as about 3 600 yards, with a minimum depth of 3½ fathoms, but with an intervening hole of 5 fathoms. The position of this easterly passage moved south from its 1860 position, the course from the Pollock Rip Light Vessel changing from due east to about southeast.

"The Broken Part of Pollock Rip has recently made out about 1 200 feet to the westward, considerably narrowing the northern entrance.

"In 1860 the Shovelful Shoal and Bearse Shoal, as defined by the 18-foot contour, were continuous and separated from Pollock Rip Shoal. In 1874 the first two of these were separated and the last two were joined together, with the southern part of Pollock Rip Shoal broken into a number of smaller shoals, which condition has continued up to the latest chart, but with varying outlines on the successive charts. The Handkerchief Shoal, which is rather more protected from the heaviest waves than the outlying shoals and therefore more nearly continuous in form, had approximately the following areas inclosed with the 18-foot curve (the dates refer to the dates of issue of charts):

"1860	1 900 acres.
1888	2 660 "
1894	2 980 "
1908	3 230 "

"The above shows a continuous increase amounting to 70% in 48 years.

"The area of water exceeding 5 fathoms in depth in the eastern extension of Butlers Hole, within which area are stationed the Shovelful Shoal and Pollock Rip Light Vessels, and limited on the west by a line drawn from the northern limit of Stone Horse Shoal to Monomoy Point, is as follows:

"1860	3 200 acres.
1888	3 000 "
1894	3 600 "
1900	3 700 "
1908	2 670 " "

The foregoing quotations show clearly the character of the shoals, how they change in position from time to time, and how, in certain areas, there is a steady accretion. The difficulties of navigating the tortuous channels, even well-lighted and marked as they are by frequent buoys and light vessels, are greatly increased by the frequently occurring dense fogs. These fogs are caused by the condensation following the contact of the warm easterly current with the colder current setting down from the coast of Maine. The Pollock Rip Light Vessel reports an average of 1 100 hours of fog occurring on 130 days per annum. So persistent and so thick are these fogs that vessels are held for days at a time at Provincetown or Vineyard Haven, unwilling to venture the passage across the shoals. The dangers are still further increased by the low-lying coast of the Cape, which becomes an exposed lee shore during east and northeast gales.

It is estimated that 22 000 000 tons of freight are carried annually around the Cape, a volume of coastwise traffic that greatly exceeds any other section of the American seaboard. It is not surprising, therefore, that the waters between Martha's Vineyard and Cape Cod Light claim the greatest toll in men, vessels, and cargo.

Fig. 1, taken from the most recent United States Coast Survey charts, shows the contour of Cape Cod, the adjacent islands, and Nantucket Shoals.

Water-borne traffic going around the Cape has the choice of two routes, either completely avoiding the shoals by passing outside of Nantucket Light Vessel, or by passing through Vineyard Sound and crossing the shoals, either through Pollock Rip, the usual course, or

south of the Great Round Shoal. Between New York and Boston the distances by these routes are:

Nantucket Light Vessel..................	408 miles.
Great Round Shoal........................	350 "
Pollock Rip..................................	342 "

By going through Hell Gate and Long Island Sound, the first distance can be reduced by 6 miles and the last two by 16 miles. The courses are shown on Fig. 1. The distance between New York and Boston *via* Hell Gate, Long Island Sound, and the Cape Cod Canal, is 264 miles.

To avoid the shoals and fogs, with their dangers and delays, projects for a trade route *via* Buzzards Bay and a canal connecting it with Cape Cod Bay have been proposed for nearly 300 years; in fact, a canal across the neck of Cape Cod has been longer under consideration than any other public work in the United States.

The first use of this route for commercial purposes was made by Miles Standish in September, 1623, when he ascended the Scusset River, a small stream that flowed into Cape Cod Bay about 20 miles south of the Pilgrim settlement at Plymouth, and, after crossing the narrow intervening low ridge of land, met the vessels of the Dutch traders from New Amsterdam, under the command of Isaac de Resieres, who had ascended the Manomet (since corrupted into Monument) River from Buzzards Bay, laden chiefly with provisions to relieve the pressing needs of the Plymouth settlers. From this beginning there was immediately established a regular traffic between the Dutch and English colonists.

As the land separating the rivers, which could be ascended easily by the small boats then in use, was only 3 miles wide, and as its elevation was less than 30 ft. above high water, it was but natural that it was soon suggested to make a through route and eliminate the portage by digging a canal.

There is a record in the quaint diary of one Samuel Sewall, under date of October 26th, 1676, that "Mr. Smith of Sandwich rode with me and showed me the place which some had thought to cut for to make a passage from the south sea to the north." In 1697 the project received official recognition, as the General Court adopted this resolve:

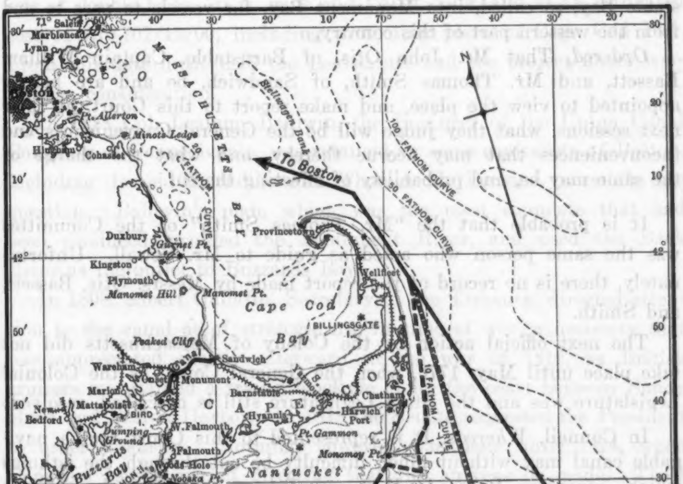

MASSACHUSETTS
Salem
Marblehead
Lynn
Boston
Pt. Allerton
Minots Ledge
Hingham
Cohasset
Duxbury
Gurnet Pt.
Kingston
Plymouth
Manomet Hill
Manomet Pt.
Peaked Cliff
Sandwich
Wareham
Onset
Monument
Marion
Mattapoisett
New Bedford
Dumping Ground
W. Falmouth
Falmouth
Woods Hole
Nobska Pt.
Buzzards Bay
Naushon I.
Barnstable
Hyannis
Harwich Port
Chatham
Pt. Gammon
Monomoy Pt.
CAPE
Cape Cod Bay
BILLINGSGATE
Provincetown
Wellfleet
Stellwagen Bank
To Boston
Nantucket Sound
MARTHA'S VINEYARD
Edgartown
Great Pt.
Nantucket
Siasconset
NANTUCKET
South West Shoal
Davis Bank
NANTUCKET SHOALS
To New York
100 FATHOM CURVE
30 FATHOM CURVE
10 FATHOM CURVE

FIG. 1.

Whereas, It is thought by many to be very necessary for the preservation of man and estates, and very profitable and useful to the public, if a passage be cut through the land at Sandwich from Barnstable Bay, so called, into Monament Bay, for vessels to pass to and from the western part of this country,

Ordered, That Mr. John Otis, of Barnstable, Captain William Bassett, and Mr. Thomas Smith, of Sandwich, be and are hereby appointed to view the place, and make report to this Court, at their next sessions, what they judge will be the General Conveniences and inconveniences that may accrue thereby, and what the charge of the same may be, and probability of effecting thereof.

It is probable that the "Mr. Thomas Smith" of the Committee was the same person who acted as guide to Mr. Sewall. Unfortunately, there is no record of the report made by Messrs. Otis, Bassett, and Smith.

The next official action by the Colony of Massachusetts did not take place until May, 1776, when the General Court, as the Colonial Legislature was and the State Legislature still is described, resolved:

In Council, *Whereas,* It is represented to this Court that a navigable canal may without much difficulty be cut through the isthmus which separates Buzzards Bay and Barnstable Bay, whereby the Hazardous Navigation round Cape Cod, both on account of the shoals and enemy, may be prevented, and a safe communication between this colony and the southern colonies be so far secured,

Resolved, That James Bowdoin and William Sever, Esqrs., with such as the Hon. House shall join, or the major part of them, be a committee to repair to the town of Sandwich, and view the premises, and report whether the cutting of a canal as aforesaid be practicable or not. And they are hereby authorized to employ any necessary surveyors and assistants for that purpose.

This Committee appointed Mr. Thomas Machin as Engineer, and undertook to prepare, probably for the first time, a survey and definite plans. Mr. Machin had scarcely entered upon his labors when he was called to other duty by George Washington, who wrote to the Chairman of the Committee:

"The great demand we have for engineers in this department has obliged me to order Mr. Machin hither to assist in that branch of the business."

After the Revolution, in 1791, the Commonwealth of Massachusetts appointed another committee to examine and report. This Com-

mittee employed James Winthrop and John Hills to make surveys and plans, and these engineers laid out a canal, practically on the route of the present one, using the Monument River, 4 rods wide, with three sets of double locks, 30 ft. wide and 120 ft. long, and at an estimated cost of £70 707/10/00, including protecting piers in Cape Cod Bay. From this date until 1818 the Legislature had the project under continuous consideration.

In 1818 Col. Loammi Baldwin, the Engineer of the Union Canal Company of Pennsylvania, was retained by some capitalists of Boston, including Israel Thorndike and Thomas H. Perkins, to study the question. Baldwin's plan, which was the most complete that had been produced, avoided the Monument River, and used the Back River as the outlet to Buzzards Bay.

In 1808, Albert Gallatin, Secretary of the Treasury, directed attention to the canal as of strategic use in time of war, a necessity that was appreciated shortly afterward in the war of 1812, as British cruisers maintained a trying blockade along the coast between Sandy Hook and Boston Harbor. In 1818 the Senate requested the President to order a survey. Nothing, however, was done until 1824, when Congress passed an act directing that full surveys be made. These surveys were in the charge of Maj. P. H. Perault, U. S. Topographical Engineer, who reported in 1825—a report which was further examined and approved by the Board of Internal Improvements for the State of Massachusetts. This report was ordered printed by Congress in 1830.

Of course, all previous plans described a canal of very small dimensions, and the canal projected by the United States was undoubtedly considerably larger than any that had been contemplated before. Even this canal, however, according to modern standards, was quite a small affair. It was to have a bottom width of 36 ft., a water surface of 60 ft., and a depth of 8 ft. Locks were planned.

Four different arrangements of locks and levels were considered, but the one finally recommended for adoption was a single level, 8 miles 524 yd. long, with the bottom on the plane of low tide in Barnstable (Cape Cod) Bay, with a tidal lock at each end. These locks were to have a length of 107 ft. and a width of 26 ft. The cost was estimated at $669 522, in which bridges were put down at $2 000. The route selected was the same as that proposed by Col. Baldwin, using Back River. Water to supply the summit level was to be furnished by

the rise in tide at the Cape Cod Bay end, admitted through regulating sluices. The Board felt that Herring Pond and its tributaries would not supply sufficient water to permit thirty-six passages per day, the estimated possible number.

With locks, the variant in the route from the Monument to Back River, as proposed by Col. Baldwin, and the Board of Internal Improvement, is feasible, but the resulting advantage and economy are not apparent.

At the time it was confidently expected that the work would be undertaken at once. When Maj. Guillaume Tell Poussin, the eminent French engineer, at one time an officer in the American Army, wrote his celebrated report on "Travaux d'Améliorations Intérieures Projetés ou Exécutés par le Gouvernement Général des États Unis", published in 1834 after his visit to this country in 1831, he described the canal as one of the great pieces of public work about to be undertaken.

The project then lay dormant for 30 years, until 1860, when it was revived by the Governor of Massachusetts calling attention to it in his annual message. The Legislature appointed a committee which reported in favor of employing engineers to restudy the matter thoroughly, as a canal seemed both feasible and desirable. The Legislature adopted the suggestion, and appointed such a committee with powers, and again another committee in 1861, which committees united in a report in November, 1862, printed as Public Document No. 41, in 1864. This report is of great value, as it reviews the whole history of the enterprise and gives many statistics of traffic and other matters having a bearing.

As to surveys and plans, the Committee availed itself first of a suggestion of Professor A. D. Bache, Superintendent of the United States Coast Survey, to make use of officers of that service. The Commissioners for Boston Harbor, the late Joseph G. Totten, Hon. M. Am. Soc. C. E., Brigadier-General, U. S. Topographical Engineers, the late A. D. Bache, Hon. M. Am. Soc. C. E., U. S. Coast Survey, and Commander C. H. Davis, U. S. Navy, Superintendent of the Naval Academy, made a report to the Committee, giving recommendations as to locks and breakwaters in Cape Cod Bay and a most important analysis of the tidal conditions, based on observations made by the late Henry Mitchell, M. Am. Soc. C. E., then an assistant in the Coast Survey.

. On receipt of this preliminary report the Committee engaged Mr. George R. Baldwin to complete the surveys and make plans. Mr. Baldwin accepted as his location the Back River route, following Col. Loammi Baldwin and the Board of Internal Improvement, but the cross-section was much greater than anything hitherto proposed, having a bottom width of no less than 120 ft. and a depth of 18 ft. Two locks were contemplated, one at each end, with two chambers in tandem, 200 and 132 ft. long, or with a combined usable length of 350 ft., 96 ft. wide. The estimated cost varied from \$9 558 000 to \$9 915 000, according to variations in details, but including the locks, changing the railway, and three large breakwaters.

The Committee found that about 10 000 vessels passed around the Cape each year, carrying miscellaneous cargo, of which coal contributed 370 827 tons in 1859. Between 1834 and 1859, both years inclusive, there had been 827 marine disasters on the Cape, involving 4 steamers, 40 ships, 71 barks, 191 brigs, 492 schooners, and 29 sloops, the average annual value of the loss being nearly \$600 000. The Committee estimated that the annual saving to navigation resulting from the construction of the canal was \$1 543 375, on the basis of 45% of the traffic using it.

The interesting feature of the report, however, was the first recorded appreciation of a canal without locks, and apparently this suggestion came from the Committee and not from any of its professional advisers, all of whom in their reports discussed locks only. The Committee stated:

"The peculiarities attending the operation of the two tide waves upon the coast has in every instance suggested to the engineers the necessity of using locks for this canal. * * *

"If some plan could be devised to overcome the force of the currents, and thereby form a free channel for the transit of vessels through from bay to bay, there could hardly be a difference of opinion upon the propriety of constructing the proposed passage, and the question whether the currents can be controlled in any way than by locks, deserves some consideration. This question was not particularly examined while the U. S. Commissioners were connected with the surveys and soundings, and their public engagements since that time have deprived the Committee of their judgment and advice upon this branch of the subject.

"* * * It may not be unreasonable to suppose that some method of avoiding the force of the currents might be discovered without the cost and delay of using locks and gates."

This vague suggestion of the Legislative Committee was crystallized into form in 1870 by Brevet Maj.-Gen. J. G. Foster, Lt.-Col. of Engineers, U. S. A., who pointed out, in a report to the Chief of Engineers, that although there was a considerable and varying difference in head at the ends of the canal, nevertheless the resulting current would not be sufficient to require locks. This contribution of General Foster's changed completely the whole character of the enterprise, because, after the publication of his report, a canal with locks was not again considered. He recommended a canal with dimensions much greater than had been previously contemplated, with a bottom width of 198 ft. and a depth of 23 ft. at mean low water.

Gen. Foster's report performed another service. It showed the great volume of traffic passing around the Cape, which could be accommodated and accommodated only by a waterway free from artificial obstructions, and, appearing as it did at the period following the war between the States, when all projects for increased transportation facilities were being eagerly taken up, it directed the attention of capitalists and promoters to the possibilities of the canal. Although no further efforts were made by either the Federal or State Governments to construct the canal, efforts by groups of financiers under private charters were continuous from 1870, the date of Gen. Foster's report, to the actual construction of the canal as described in this paper.

In 1870 a charter was given by the State to the Cape Cod Ship Canal Company, among whose incorporators were Alpheus Hardy, Thomas Russell, Charles H. Allen, Rufus Ingalls, and Charles A. Secor. This charter was regarded with favor by committees of Congress and the Legislature of the State, but nothing was accomplished, and at length it was allowed to lapse, after it had been extended by the Legislature several times. The company referred the tidal questions to Clemens Herschel, Past-President, Am. Soc. C. E., who confirmed the previously expressed opinion of Gen. Foster that the resulting current in a sea-level canal would not be sufficiently swift to prevent passage, and that a lock was not only unnecessary, but detrimental.

Although Massachusetts at an early date provided for the construction of railroads by general legislation, thus doing away with special legislative charters, no such provision was made in the general

laws concerning the construction of canals, so that recourse to the Legislature for powers has always been necessary.

When the charter to the Cape Cod Ship Canal Company lapsed in 1880, a new one was granted to Henry M. Whitney, William C. Whitney, Henry F. Dimock, Charles T. Barney, Holcomb Hosford, and associates, for the Cape Cod Canal Company. These gentlemen investigated the subject, retaining George S. Greene, Jr., M. Am. Soc. C. E., as Consulting Engineer, but they, too, permitted their charter to lapse.

In 1883 another act was passed by the Legislature, incorporating the Cape Cod Ship Canal Company, with a capital stock of \$5 000 000, the persons named being William Seward, Jr., George S. Hall, Samuel Fessenden, Edwin Reed, William A. Clark, Jr., Joseph T. Hoile, Walter Lawton, William F. Drake, and William Parker. This company made a contract with Frederic A. Lockwood to construct the canal. Mr. Lockwood was a singular genius. At one time he was a Baptist minister, but, being of a mechanical turn, he established a machine shop in East Boston, and there designed a curious type of suction dredge, under the patents of one John A. Ball of California. Being a man of much force and power of persuasion, he organized and procured the charter for the company just mentioned, primarily in order to give work to his machine shop and create an opportunity to use his dredge, he having acquired from Mr. Whitney and his associates their plans and surveys and whatever rights they possessed. He appointed Mr. George H. Titcomb Chief Engineer and Mr. Charles M. Thompson Assistant Engineer. The latter gentleman remained at Sandwich even after the Lockwood efforts came to an end, assisting the subsequent companies, and became Real Estate Agent of the present company, which position he held until his death in March, 1914.

Now, for the first time since Miles Standish began the trading route in 1623, and after all the fruitless surveys and plans by the United States, by the Commonwealth of Massachusetts, and various private parties, actual work was begun. Lockwood built his dredge, and with it cut through the open beach just north of Sandwich and near the mouth of the Scusset River. In order to obtain the necessary funds to carry on the work, Lockwood succeeded in persuading Mr. Quincy A. Shaw, of Boston, to advance them. With this aid, Lock-

wood and his singular excavating machine made a channel nearly a mile long, about 15 ft. deep and perhaps 100 ft. wide through the sandy marshes of the Scusset. While thus at work, Lockwood suffered a stroke of apoplexy, completely disabling him. He then conveyed to Col. Thomas L. Livermore, of Boston, as Trustee, all title in the chartered company, canal, land, and dredge, as security for Mr. Shaw's advances and some minor obligations. With further capital advanced by Mr. Shaw, the Trustee continued dredging for several months, carrying the excavation of the canal to a total of about 700 000 cu. yd., and acquiring title in fee to land which, with that purchased by Mr. Lockwood, amounted to about 1 000 acres. Then, probably realizing the hopelessness of completing the work with a single dredge, and especially with such a dredge as the one in hand, the Trustee stopped work, and the dredge was subsequently and mischievously set on fire and completely destroyed. The action of the waves and littoral drift closed the entrance through the beach and filled perhaps one-half of the excavation with sand.

If little physical result was accomplished, the Lockwood attempt is entitled to the honor of the first actual construction and a demonstration that some people were at length willing to do more than make surveys, and great credit must be given to Mr. Shaw and his Trustee, Col. Livermore, for having perfected the titles to so many parcels of land, and especially for keeping them intact and free from physical encumbrances that would have prevented the construction of the canal. Other routes, and variations of the route adopted by Lockwood's company, have been considered, but the one he selected —that since adopted—is the only one that is feasible. Had the land passed back into the hands of its many original holders, its re-acquisition would have been difficult and perhaps so expensive that the cost would have been prohibitory; or had it been "improved", canal construction would have been impossible. When the present company took up the work, the fact that more than 80% of the right of way could be acquired in fee simple at a single purchase, and at an ascertained reasonable cost, contributed in no small degree to the favorable consideration of the project. Other men less far-sighted than Col. Livermore would not have had the courage to keep the holdings together, and the failure to do so certainly would have jeopardized the realization of the canal, if not actually preventing it.

Following the cessation of work under the Lockwood charter, peace reigned for a few years, broken by the passage in 1891 of a charter to the Boston, Cape Cod and New York Canal Company, under which, however, nothing was done.

In 1893 the Legislature acted again, this time in granting a charter to the "Old Colony and Interior Canal Company", among whose incorporators were two prominent contractors, James D. Leary and Warren Roosevelt, and an energetic attorney of New York, William G. Bussey, of whom more later. This charter, which for the sake of safety repealed in terms all prior charters, provided for a choice of routes *via* the Monument or Bass Rivers, the latter flowing into Vineyard Sound. The Bass River route would have given a shorter canal, but with a less saving in distance, and with a failure to avoid fogs and very bad tidal currents.

These gentlemen did nothing with their grant, as likewise Oliver Ames, of Boston, and associates with a charter to the Massachusetts Ship Canal Company passed in 1895.

The Massachusetts Maritime Canal Company was the next step, chartered in June, 1896, the projector being Mr. William G. Bussey, and one of the incorporators being the late Elmer L. Corthell, Past-President, Am. Soc. C. E. The charter called for a canal of increased dimensions, *viz.*, a depth of 25 ft. at mean low water and a bottom width of 100 ft. Full and complete engineering and commercial investigations were made by Mr. (later Dr.) Corthell and the late Alfred P. Boller, M. Am. Soc. C. E. Mr. Bussey, ably assisted by Mr. Corthell, made every effort to secure the necessary capital, interesting in the project men like Myron T. Herrick of Cleveland and Lewis Nixon of New York, but finally they were obliged to let the charter lapse.

The next, and as it proved the last, step, was an application by Mr. DeWitt C. Flanagan, in 1899, for a charter, which was passed on June 1st of that year, incorporating the Boston, Cape Cod and New York Canal Company. The incorporators named in the act are Alexander Dow, David W. Belding, Charles E. Hoge, Richard G. Peters, Thomas F. McGarry, Walter Clifford, Charles H. Phelps, DeWitt C. Flanagan, and William O. Brown. Mr. C. C. Dodge was elected President of the company, holding the office until his death in 1910.

The company appointed Dr. Corthell its Engineer with Mr. Charles M. Thompson in charge on the ground. Arrangements were made with the Maryland Trust Company of Baltimore to assist in the financing, and that company appointed the late Alfred L. Rives, M. Am. Soc. C. E., formerly Colonel, C. S. Engrs., as engineer in its behalf. Col. Rives, a member of the Virginia family of that name, and formerly an officer in the Confederate Army, had been for some years Superintendent of the Panama Railroad for the French company that owned the railroad and was building the Panama Canal. Dr. Corthell and Col. Rives acted as joint engineering advisers, rendering valuable assistance in the preparation of plans and in various hearings before the State authorities. Then Mr. Bussey re-appeared on the scene and, as counsel, prepared an operating plan for carrying out the work. Unfortunately, however, unfavorable financial conditions arose, the Maryland Trust Company became involved, Mr. Bussey and Col. Rives died, and Mr. Flanagan, in order to prevent the charter from lapsing, was compelled to use his own private means to make the deposit of $200 000 with the State Treasurer and $25 000 with the County Treasurer, as required by the charter, to insure payment for land expropriated and claims for damages.

Finally, in 1904, Mr. Flanagan laid the project before Messrs. August Belmont and Company of New York, who promised to take it up when the general financial outlook should brighten.

Before considering the charter and describing the canal, it should be remarked that though there is only one feasible route for a canal across Cape Cod, as stated before, other routes for a canal westward from Massachusetts Bay, of quite different character and location, have been proposed. Of these, the one most persistently advocated was a canal from Fore River, near Boston, *via* Brockton, Bridgewater, and Taunton to Narragansett Bay. Such a canal and the others on similar inland routes were barge and not ship canals. They necessarily called for large investment and required many locks, and as the water in the central portion would have been fresh, such canals would have been inoperative—like the Erie Canal—during the winter. Full plans and estimates of these canals have never been prepared.

The act incorporating the Boston, New York and Cape Cod Canal Company is known as Chapter 448, Acts of 1899, and has been amended by Chapter 476 of the Acts of 1900 and Chapter 519 of the Laws

of 1910. These three acts constitute the charter of the Canal Company, from which it derives all its powers and rights. This charter provides:

1.—That the company may issue its capital stock to the extent of not exceeding \$6 000 000 and bonds to the extent of \$6 000 000;

2.—Right to construct and operate a canal from Cape Cod Bay to Buzzards Bay, with all structures, wharves, docks, breakwaters, etc., convenient for the canal, and operate steam and other vessels;

3.—That the canal, including its approaches in the open waters of Cape Cod and Buzzards Bays, shall have a minimum depth at mean low water of 25 ft., a minimum width on the bottom of 100 ft. at that depth, side slopes of not steeper than 2 horizontal to 1 vertical, and consequently a minimum water surface of 200 ft.;

4.—Powers for taking land and liability for damages similar to those of railroads;

5.—Obligation to reconstruct the portion of the Old Colony Railroad (leased to the New York, New Haven and Hartford Railroad) where affected by the construction of the canal, including a bridge or tunnel across the canal;

6.—Obligation to provide and maintain without charge, ferries, bridges, or tunnels for highways;

7.—Obligation to construct highways to connect with the crossings and to replace those destroyed by the canal;

8.—Obligation to deposit with the Treasurer of the Commonwealth \$200 000 and with the Treasurer of Barnstable County the sum of \$25 000 as guaranties that land damage claims will be paid;

9.—Power to charge tolls for the use of the canal and for towing at such rates as the directors may determine;

10.—Punishment for wilful damage of the canal by payment to the company of treble the amount of damage sustained and by a fine not exceeding \$1 000 or imprisonment for not exceeding one year;

11.—Official control by the:

Harbor and Land Commission as to approval of general plans;

Railroad Commission (now the Public Service Commission) as to:

Relocation of Old Colony Railroad;

Acceptance of bridge or tunnel for crossing of the railroad over or under the canal;

Fixing of rules for operating said bridge;

Joint Board, composed of the above two boards sitting as a single board, as to:

Issue of capital;

Point of crossing the canal by the Old Colony Railroad;

Method of crossing the canal by highways, whether bridge, tunnel, or ferry;

General supervision of the work;

County Commissioners of Barnstable County and Selectmen of the towns passed through, as to:

Relocation of highways;

Points of highway crossing of the canal;

Condemnation of property.

The charter also provided that the Harbor and Land Commissioners, the Railroad Commissioners, or the Joint Board may employ an engineer or engineers whose compensation shall be paid by the Canal Company.

In 1909 Mr. Belmont decided to begin construction, and, in order to provide the necessary legal machinery through which financial arrangements could be made, he organized the Cape Cod Construction Company, which, with the consent of the Railroad Commission of Massachusetts, took the contract to construct the canal for the amount of the bonds and stock authorized by the charter, namely, bonds bearing interest at 5% to the par value of $6 000 000 and 59 900 shares of stock with a par value of $100 each, 100 shares of stock having been previously authorized and issued to the incorporators.

The directors and officers of the Cape Cod Construction Company, since the work began, have been August Belmont, Charles H. Allen, F. R. Appleton, E. Mora Davison, A. L. Devens, DeWitt C. Flanagan, W. A. Harriman, E. W. Lancaster, L. F. Loree, Jacob W. Miller, William Barclay Parsons, F. DeC. Sullivan, Frederick D. Underwood, and H. P. Wilson. The executive officers have been Mr. Belmont, President; Messrs. Miller and Devens, Vice-Presidents; Mr. John J. Coakley, Treasurer, and Mr. U. A. Murdock, Secretary.

On the completion of the canal (when the contract between the Canal Company and the Construction Company was declared completed), the above gentlemen, except Messrs. Devens and Lancaster, who died while the work was in progress, and Mr. Davison, became directors of the Boston, Cape Cod and New York Canal Company in charge of

operation, with Mr. Belmont as President, and Mr. Miller as Vice-President and General Manager.

Prior to action by Mr. Belmont, Mr. Flanagan and his associates had had surveys made and plans prepared under the direction of Dr. Corthell and Col. Rives. When the Cape Cod Construction Company was formed, the writer was appointed Chief Engineer of that company and of the Canal Company as well, and made a new survey of the route. The location finally adopted was substantially that made by Messrs. Corthell and Rives, except that, in actual construction, the center line of the canal was placed generally 50 ft. north of and parallel to the center line of the right of way, so that a canal with a bottom width of 200 ft. could be constructed by widening on one side only for the most part. This additional construction would place the axis of the canal coincident with that of the right of way. Certain modifications in the location of the approach channel in Buzzards Bay from the original Corthell-Rives plans were also made as the result of further study subsequent to their surveys.

The location as thus determined commenced in the open waters of Cape Cod or Barnstable Bay, off the unbroken sand beach, about 3 miles north of the Village of Sandwich and 20 miles south of the City of Plymouth. It then traversed the low-lying marshes of the Scusset River and crossed the land forming the divide between Cape Cod and Buzzards Bays, passing through a depression at the Village of Bournedale, where the surface of the ground on the location had an elevation of only 30 ft. above mean sea level, although it rose, after a level width of about 1 000 ft., occupied by the location, to an elevation of about 125 ft. on both sides. Then the location followed the general line of the Monument River, the depth of which varied from 1 to 4 ft., to Buzzards Bay. In Buzzards Bay the existence of shallow shoals with many boulders prevented (except at prohibitive cost) the construction of a straight channel from the mouth of the Monument River to deep water in Buzzards Bay off Wings Neck Light, so the location as adopted followed the general line of the existing natural channel, which required two turns.

The details of this location, with a profile of the surface along the center line of the canal, are shown by Fig. 2. A line drawn from the center of the canal at the mouth of the Monument River

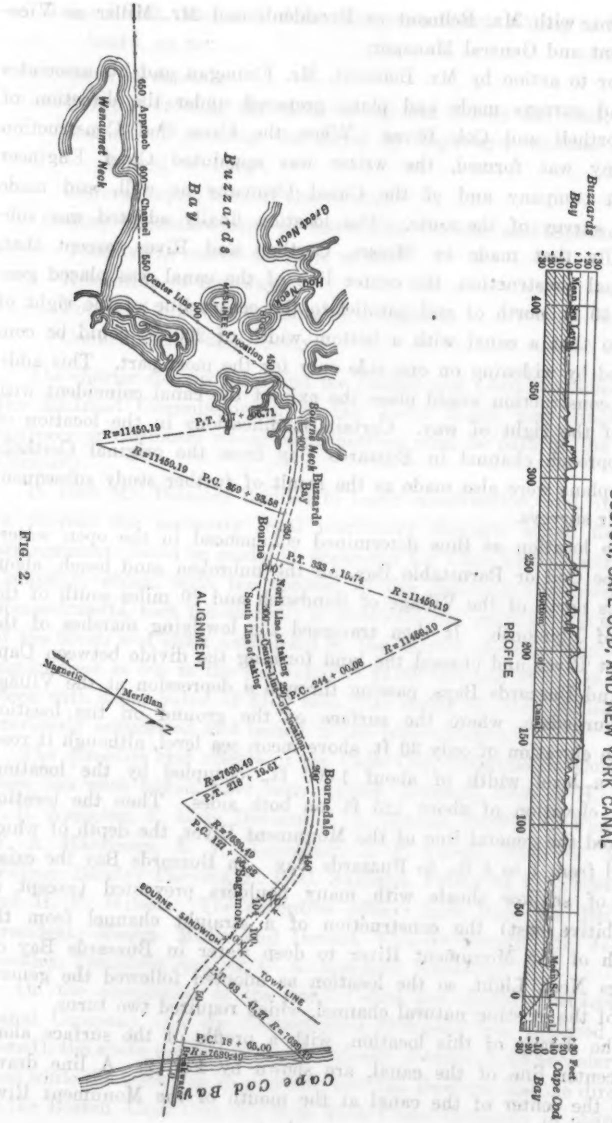

BOSTON, CAPE COD, AND NEW YORK CANAL
PROFILE
ALIGNMENT
Buzzards Bay
Cape Cod Bay
Wenaumet Neck
Great Neck
Bourne Neck
Bourne
Bournedale
Sagamore
BOURNE - SANDWICH TOWN LINE
North Line of taking
South Line of taking
Magnetic Meridian

FIG. 2.

to the center of the canal at the Cape Cod Bay end lies almost exactly due east and west (magnetic). The canal, therefore, was considered as running east and west, and for convenience this was considered diagrammatically true of the Buzzards Bay approach, although the actual variations are considerable.

The portion of the canal under the jurisdiction of, and control by, the State, and covered by the charter, extends from the beach line of Cape Cod Bay, known as Station 11 + 70, to the mouth of the Monument River, known as Station 417 + 18.4, both being stations of the canal survey. For such distance, 40 548.40 ft., the company owns a right of way 1 000 ft. wide for 9 230 ft., at the east end, and 600 ft. wide from Station 104 + 00 to Station 417 + 18.4. This length is referred to as the "canal proper", being the limits of the portion belonging to the company and over which tolls can be collected. The approaches in Cape Cod Bay and in Buzzards Bay beyond these limits are in open waters, over which the United States maintains jurisdiction and for which there is no right of way or charter, excavation having been done under a permit from the War Department.

The original plans of Messrs. Corthell and Rives contemplated three passing places, with an extra bottom width of 100 ft., which, after discussion with the engineer of the Joint Board, were concentrated at the ends, so that the canal as built, out of a total length of 68 600 ft. has only 30 800 ft., or less than 6 miles out of 13 miles, with the minimum width of 100 ft. on the bottom. In the canal proper, changes in direction are effected by curves with radii varying from 7 589.49 ft. (1.5 miles nearly) to 15 113.04 ft. (2.86 miles), the longest curve being 9 522 ft.; and in Buzzards Bay by the tangents meeting with deflection angles without curves, a straight-line widening being made on the inside of the angles to give vessels swinging room in turning.

The chief features of the location are as follows:

	Length: In feet.	In miles.
Canal proper (Stations 11 + 70 to 417 + 18.4) .	40 548.40 =	7.68
Dredged approach, Cape Cod Bay (Stations 0 — 14 to 11 + 70)	2 570.00 =	0.49
Dredged approach, Buzzards Bay (Stations 417 + 18.4 to 672)	25 481.60 =	4.83

	LENGTH: In feet.	In miles.
Total dredged waterway (Stations 0 — 14 to 672)	68 600.00 =	13.00
Tangents in canal proper	12 342.66 =	2.34
Tangents in Cape Cod Bay	2 570.00 =	0.49
Tangents in Buzzards Bay	25 493.29 =	4.83

Number of curves in canal proper....................4

Length of curves in canal proper:

Radius, in feet.	Length, in feet.
{ 3 536.25 { 1 429.81	{ 7 589.49 to 7 639.49 { 11 509.19
9 009.33	7 639.49
{ 6 999.92 { 2 522.28	{ 11 409.19 { 15 113.04
5 773.33	11 459.19

Total length of curves in canal proper: 29 270.92 ft. = 5.54 miles.

Number of angular turns in Buzzards Bay....................2

Deflection angles of turns: Station 430..............43° 14′

" 540..............50° 29′ 41″

Length with bottom width of 300 ft.	4 400 ft.
" " " " " 250 "	27 200 "
" " " " " 200 "	3 000 "
" " " " " 150 "	1 500 "
" " " " " 100 "	30 800 "
" " " " " 250 to 450 ft. (turns)	4 000 "
" " " " " 200 to 300 " transition	1 000 "
" " " " " 150 to 250 " "	300 "
" " " " " 100 to 200 " "	400 "

Slight apparent discrepancies will be noted in some of the foregoing figures. The explanation is that the total lengths of the canal are given as measured on the center line of the location, but the detailed lengths of the curves and tangents are those of the present canal, as actually constructed, offset from the center line of the right of way. When the canal is widened and the two center lines become coincident, the summation of the lengths of curves and tangents will agree with the total lengths as given, and the varying lengths of the radii of the compound curves will disappear.

The adopted cross-sections of the canal are shown on Fig. 3, in which the steepest slope in the canal proper is put at 1 on 2 to a point 6 ft. above high-water level; in the approaches, however, where wave action is more pronounced, and no opportunity is afforded to protect the slopes against wave action, the minimum slopes were made

1 on 3. As there is considerable difference in tidal elevation at the two ends, the mean amplitude of the tide at the eastern end being nearly 10 ft., the bottom grade of the canal is on a slope in order to give a depth of 25 ft. at mean low water. Thus the elevation of the bottom of the canal at the east end is 30 ft. below mean sea level, and at the west end it is 27.5 ft. The slope of the bottom, however, is on a curved, and not a straight, line, as the heights of mean high and mean low water in the canal proper make concave and convex curves, respectively, all of which details will be referred to later.

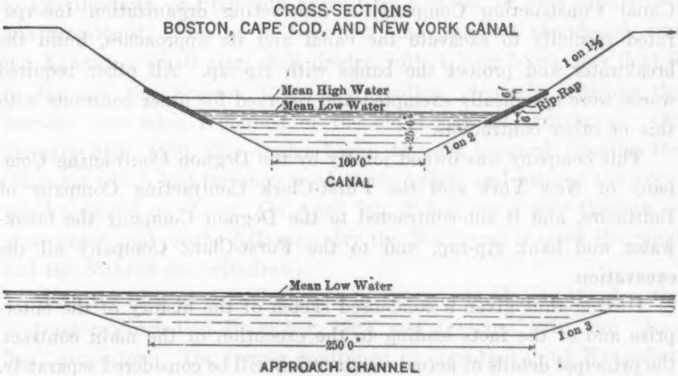


FIG. 3.

At the west end the approach and canal entrance are land locked and required no special consideration, but at the east end the canal debouches on an open sandy shore, with full exposure to winds coming from any part of the quadrant north to east. Winds west of north (magnetic) are broken by the high ground south of Plymouth, and winds east of northeast (magnetic) are partly broken by the low-lying cape 25 miles distant and by the land constantly getting nearer as the bearing approaches south, when the land makes a complete lee. Between north and northeast the winds have a clean sweep from the Maine coast, a fetch of 180 miles.

To protect the easterly entrance to the canal, and permit vessels to enter at times of storm, there were designed and constructed two parallel breakwaters, 800 ft. apart. The larger one, on the north side, has a length of 3 000 ft. from the contour of mean high water, and

provides protection against north and northeast winds. The smaller, 1 000 ft. long, on the south side, is intended to stop the littoral sand movement from the south.

Other works required were: the protection of the slopes with rip-rap from 6 ft. below mean low, to 6 ft. above mean high, water, the relocation of the railway, the construction of certain bridges and ferries, highways, lighting system, and other aids to navigation, all of which will be described in order.

The surveys and plans being complete, a contract was made on May 15th, 1909, after competitive bidding, with the Degnon Cape Cod Canal Construction Company, a contracting organization incorporated specially to excavate the canal and its approaches, build the breakwater and protect the banks with rip-rap. All other required works were specifically exempted and reserved for other contracts with this or other contractors.

This company was owned jointly by the Degnon Contracting Company of New York and the Furst-Clark Contracting Company of Baltimore, and it sub-contracted to the Degnon Company the breakwater and bank rip-rap, and to the Furst-Clark Company all the excavation.

Having thus given a condensed sketch of the history of the enterprise and of the facts leading to the execution of the main contract, the principal details of actual construction will be considered separately.

Excavation.—The work was formally begun by Mr. Belmont "turning the first sod" on June 22d, 1909, though actual construction was started on June 19th by the Degnon Contracting Company depositing the first stone in the breakwater. The contractors recognized that the breakwater must be advanced so as to afford some lee for a dredge to cut through the beach of Cape Cod Bay.

The Furst-Clark Company, dredging contractors of long and varied experience, considered that the excavation, certainly of the canal proper, could be done almost wholly by hydraulic dredges, and arranged for such plant, except in the approach channel in Buzzards Bay, where, spoil area for pumping not being available, the material had to be dug and removed in scows. They acted promptly, placing the dredge *Kennedy* at work in Buzzards Bay on August 2d. This dredge was of the "ladder" type with $\frac{5}{8}$-cu. yd. buckets; it was followed on October 25th by *Coastwise Dredge No. 1,* a small "clam-shell"

machine. The plan was that these dredges, to be assisted by others later, were to cut a deep channel through to the Monument River to permit hydraulic dredges to enter and excavate the canal proper while excavators of other types were completing the approach channel.

On October 16th the first attempt to cut through at the east end was begun by the *Mackenzie,* a 22-in. hydraulic dredge, starting as close to the beach as it could work. The attempt was not successful. As the breakwater, in its early stage, provided but little protection, and as the waters were exposed, the dredge had to be withdrawn either to Provincetown or Plymouth every time a storm, or even high wind, was threatened. Finally, in December, the plan was abandoned, and the *Nahant,* a small clam-shell dredge with a long boom, was floated in through the Scusset River and shallow channels crossing the marshes into what remained of the old Lockwood excavation. On January 24th, 1910, this dredge began to work seaward, opening the old canal which had become closed at the beach, and placing the excavated material on shore. On April 7th, it had cut a way through to open water, and on the following day the *Mackenzie* entered the canal and the *Nahant* was withdrawn.

With the idea of removing some of the over-burden in the dry, the contractors erected in December, 1909, and January, 1910, two "drag line" excavators. The former continued in operation until November, 1910, and the latter until February, 1911. The results were very unsatisfactory, as these machines were not adapted to the soil to be excavated, and were too heavy to move over rough and swampy ground. They involved the contractors in a heavy loss.

In the spring of 1910 the Furst-Clark Company placed at work another 22-in. hydraulic dredge (known as *Number 9*) at the east end to work in tandem with the *Mackenzie,* and at the west end the 4-cu. yd. dipper-dredge, *Bothfeld,* followed in August by the *Onondaga,* a 9-cu. yd. dipper-dredge, and in September by a smaller machine, the *Neponset.* In July, the *Bothfeld* having deepened the channel sufficiently, the *Warren,* a 12-in. hydraulic dredge, entered the Monument River and began spoiling on shore.

By this time the contractors realized that they faced a much more serious task than they had anticipated. The material contained so many boulders and such a high proportion of cobbles, cemented gravel, and stiff clay that it could not be excavated completely by hydraulic

dredges except in places, and recourse must be had to "dipper" machines. The contractors thereupon purchased the plant of another company, consisting of three dipper-dredges: the *National, Capitol,* and *International,* with buckets of from 5 to 6 cu. yd. and the *Federal,* a small hydraulic dredge. The first three arrived in Buzzards Bay in November and December, 1910, but the *Capitol* and *National* were at once towed around the Cape to the east end to assist the *Mackenzie,* which was having difficulty with the hard material, the *Number 9* having been already withdrawn.

At this time there were at work one first-class hydraulic dredge, one old ladder-dredge, in good condition but not adapted to this work, one small hydraulic dredge, and five dipper-dredges, all old and of designs fitted for digging only soft material, and it was evident that the plant, even as reinforced by the purchase of the hydraulic and the three dipper-dredges, would not suffice. Arrangements were then made to remove some top material by steam shovels, work which the excavators had failed to do. Two shovels, with 2 and 2½-cu. yd. buckets, were set to work during the summer of 1911, when the *Federal* started at the west end of the canal, and the *Suffolk,* another 12-in. hydraulic, took the place of the *Warren.*

The contractors were soon compelled, however, to realize that their dipper-dredges, or in fact any other dipper-dredges then in existence, were wholly unsuited to the allotted tasks, and that, if they were to complete the work, powerful modern equipment must be obtained. Then, at the end of 2 years, instead of at the outset, orders were given for the construction of two powerful dipper-dredges, the *Governor Warfield* and the *Governor Herrick.*

These dredges were designed and built by The American Locomotive Works, of Paterson, N. J., being advised by A. W. Robinson, M. Am. Soc. C. E., and Mr. A. D. Morris, Mechanical Engineer of the Furst-Clark Company. The hulls are of steel, 135 ft. long, 42 ft. wide (though 50 ft. wide over the forward spud sponsons), 7 ft. draft when resting on the spuds, and 10 ft. maximum draft when floating. The forward spuds are 70 ft. long and 42 in. square in cross-section, with "pin-up" and "pick-up" operation by cable. The stern spuds are 70 ft. long and 30 in. square, with rack and pinion pick-up. All spuds are made of braced steel plates. The forward spuds are placed so that the boom can swing through an arc of 180°, a necessary feature

when a dredge is to work in a heading and load scows alongside. The boom, 57 ft. long, is of the rigid Robinson model. The dipper-handle, of composite steel and wood construction, is 2 ft. 6 in. by 2 ft. 8 in. in cross-section, and 62 ft. 10 in. long, capable of digging in 40 ft. of water. On the boom, controlling the dipper-handle, is a "crowding" engine, which is controlled by the dipper tender.

For each operating part of the dredge there is a separate engine, as follows:

1 Main hoisting engine; two 18 by 24-in. cylinders; single condensing.
1 Backing drum, operated by main engine.
1 Swinging engine; two 10 by 14-in. cylinders; single condensing.
2 Forward spud engines; two 12 by 15-in. cylinders; single condensing.
2 Aft spud engines; two 8 by 8-in. cylinders; single condensing.
1 Crowding engine; two 12 by 15-in. cylinders; non-condensing.
1 Dipper latch engine; 7 by 24-in. cylinder; non-condensing.
2 Deck engines; two 9½ by 12-in. cylinders; single condensing.
2 Capstan engines; two 6 by 8-in. cylinders; single condensing.
1 Rock hoist engine; two 8 by 12-in. cylinders; single condensing; capable of lifting 60 tons.
1 Surface condenser; 1 500 sq. ft. cooling surface.
1 Refrigerating plant.
1 Electric light engine; 10 kw.
1 Air compressor.
5 Pumps.
2 Scotch marine boilers, 116 in. in diameter, 7 ft. long; 1 350 sq. ft. of heating surface, capable of generating steam at 135 lb. pressure.
1 Auxiliary vertical boiler; 48 in. in diameter, 9 ft. high.

The dippers are of two sizes, 10 cu. yd. for sand, gravel, etc., and 8 cu. yd. for rock. The main hoisting cable is a single line, 2¼ in. in diameter. The backing spud and swinging cables are 1¾ in. and 2¼ in. in diameter, respectively. By avoiding sharp turns in the cables and using large sheaves where turns were necessary, a very high average life of cable is secured. The main and backing cables will dig about 300 000 cu. yd., scow measure, of hard material. The spud cables will last for about 1 year of continuous work for the "pin-up" and nearly 2 years for the "pick-up" cables; the swinging cables will last from 5 to 7 months.

The vessels are equipped with quarters for double crews, including an inspector's room, and a large galley with separate messrooms for officers and crew.

These dredges were built on the canal. The plates for the hull were sent to the site punched, and, after the hulls had been constructed and launched, the several engines, built at Paterson, were shipped in pieces and erected on board.

After tuning up and some reconstruction, these two machines have made an extraordinary record. The best single day's record was more than 8 000 cu. yd., and the best month 131 000 cu. yd., in both cases "place" measurement, although dredges are usually rated by scow measurement, which includes a swell of from 15 to 20 per cent. These records were accomplished, not in an open seaway with plenty of room, but in a narrow channel, and in a closed heading involving the turning of nearly every scow so as to load both ends. The scows were also necessarily small, usually having a capacity of from 600 to 700 cu. yd., and the material dug was very hard sand when at best, and contained boulders running up to 30 and 40 tons. The average record, as shown by 9 consecutive months when engaged in straight work, was more than 95 000 cu. yd. per month.

The operating crew for double shift—so as to give continuous service—consists of from 25 to 28 men for two shifts as follows:

1 Captain,
1 Chief Engineer,
1 Second Engineer,
2 Operators,
2 Dipper tenders,
1 Handy man, with rank of dipper tender,
2 Oilers,
6 Firemen (3 shifts),
2 Mates,
6 Deckhands,
1 Cook,
1 or 2 Messboys.

When in continuous operation, 11 tons of soft coal are burned daily. The *Herrick* began work in July, and the *Warfield* in August, 1912.

While these dredges were being built, the Chief Engineer urged the contractors to get more plant at work in the central portion. One plan that was considered to increase the plant was to place the large hydraulic dredge *Mackenzie* on railway trucks and on a special ship railway transport it over the right of way from about Station 140 to about Station 220, where a pool in the Monument River could be formed to permit it to begin work. This plan was abandoned in favor of building a new hull for the *Federal*, and erecting in it new boilers but retaining the old pump and machinery, and converting the dredge at the same time into a 15-in. machine. This was done, and the reconstructed *Federal* was set to work inland in August, 1912. By thus starting an operating unit between the headings, the same advantage was obtained as sinking a shaft in tunnel work.

As excellent and economical as were the dredges *Warfield* and *Herrick*, they were built too late to permit the work to be completed within the limits of the contract. In August, 1912, the Construction Company made a separate contract with the E. W. Foley Construction Company for a steam shovel to assist in the removal of the overburden, and in the autumn of that year took up with the Degnon Cape Cod Canal Construction Company the whole question of carrying on the work. The outcome of this was the dissolution of that company, and the making, as of November 1st, 1912, of two contracts directly with the component partners in that company who had heretofore been acting as sub-contractors. The Degnon Contracting Company undertook to complete all the stone work, under the terms and conditions of the original contract, which they carried out. The Furst-Clark Construction Company took over, under a direct contract, but under different terms from the original contract, all the excavation except such as the Construction Company might do itself or under other contracts with other parties, including the single Foley contract already in force.

When it was first fully realized that the excavation was more difficult, not only than originally expected, but even after considerable experience, the writer urged on the general contractor the advisability of extending the operations of the steam shovels, and even to take 2 or 3 miles in the central portion of the canal proper, and not only remove the part above ground-water level, which was about Elevation 108, by shovels, as they were doing in a small way, but to

erect pumps, carry the Monument River in a flume around the work and then complete the excavation of the canal to grade and rip-rap the banks in the dry, breaking up the boulders encountered for this purpose. The contractors, however, were afraid that an excavation to a depth of 36 ft. below the level of ground-water and having a width at the bottom of 100 ft. for a distance of from 10 000 to 15 000 ft. in a soil composed chiefly of sand and gravel, would be one that could not be kept sufficiently dry, and they declined to undertake it.

When the excavation contract was remade, in December, 1912, but effective as of November 1st, the Construction Company, realizing that the Furst-Clark plant could not probably complete the work as quickly as desired, decided to undertake itself the working of steam shovels below water level, and made a new contract with the Foley Company, with profits dependent on success. In addition to its 2-cu. yd. shovel already at work, the Foley Company purchased a 2½-cu. yd. shovel, which had been working in the dry under a sub-contract with the Furst-Clark Company, which work was substantially completed.

At first a stretch of work between about Stations 235 and 255 was taken, and a steam shovel put in. This section was selected because the Monument River between these points lay in its old bed, outside of the canal excavation. It was soon found that the difficulty of maintaining a sufficiently dry excavation to a depth of at least 10 ft. below ground-water was negligible, and it was decided to extend the operations. The Foley Company arranged for a third shovel.

The Monument River flows into the canal at Station 195, coming from the north and almost at right angles to it. To the eastward of this station the ground above Elevation 108 had been removed, and as this was but little higher than the elevation of the bed of the river, a small dam was sufficient to divert its flow to the eastward, so that the only water to be encountered was ground seepage. Four natural dams were left, at Stations 208, 216, 234, and 276. It was first expected to confine operations between the dams at Stations 216 and 276, but afterward, when it was seen that the advance progress of the dredge at the east end was slower than scheduled, excavation by steam shovel was extended to the section lying between Stations 216 and 208.

The pumping plant consisted of four centrifugal pumps, with 14-in. suction and 12-in. discharge, driven by General Electric induction

motors developing 50 h. p. at 550 volts; two 5-in. centrifugal pumps, direct-connected to 550-volt induction motors; one 10-in. and one 8-in. steam pump.

At one time the whole excavation from Stations 208 to 276 (6 800 ft.) was open, and a large part with an average depth of at least 20 ft. below the level of ground-water, and the foregoing plant, which had one-half in reserve, was amply sufficient to keep the trench free of water. In fact, one of the large pumps running steadily would have sufficed.

The maximum depth made by the Foley Company was a cut at Elevation 85 (mean sea level being 100), or 23 ft. below the elevation of ground-water. Between Stations 208 and 216 the bottom of the pit was at about Elevation 95; between Stations 216 and 234 at 90; and between Stations 234 and 276 from 85 to 90. Some of the boulders encountered were broken by blasting, and the pieces were used to riprap the north bank of the excavation between Stations 220 and 265.

The Foley shovels were served by narrow-gauge locomotives and 4-cu. yd. cars.

The first of the dams to be removed by a dredge was that at Station 276, then followed the dams at Stations 208 and 216, in order, leaving that at Station 234 for the last.

The Furst-Clark Company continued using its dredging equipment, but with the addition of another old dipper-dredge, the *Weymouth,* with a 5-cu. yd. dipper, while the Foley Company operated three steam shovels by the aid of pumps under a separate contract. Thus the work continued during 1913, and by June, 1914, the only remaining excavation in sight was the Foley dam at Station 234. This was removed in July, and the formal opening, but of a canal not to full depth at all points, was on July 29th, 1914. On the following day the canal was thrown open to commercial traffic by vessels drawing not more than 15 ft.

Immediately after the opening, the Furst-Clark Company objected to continuing the work of completion with traffic having the right of way, and stopped work in September. An arrangement was then effected whereby the Construction Company undertook the work of completion, the Furst-Clark Company turning over the dredge *Warfield* and some other plant for use by the Construction Company.

In the completion of the work the Construction Company used three dredges and two corps of divers and lighters. As it was decided to maintain traffic, it was desirable that the plant used in the portion of the canal where the bottom width was 100 ft. should occupy the minimum of space. An investigation showed that much of the material in this portion could be handled by a powerful suction-dredge, and such a machine was found in *Dredge No. 3,* belonging to the Metropolitan Dredging Company, Mr. R. P. Marshall, President. This dredge had a 20-in. suction, and an engine capable of developing 1 000 h. p. It was chartered, set to work in December, 1914, and dismissed from charter on October 2d, 1915, during which time it rendered most efficient service. *Dredge A,* of the Standard Engineering Company, also a 20-in. machine, was taken under charter and used to complete the excavation at the east end. The *Warfield* worked generally in the wider portions of the canal and the approaches, although it was used in the 100-ft. section, as it was found that, with a little practice, scows could be shifted from alongside to ahead and back to permit vessels to pass.

When the *Warfield* was first used in the 100-ft. section, the dredge worked only on a fair tide, as it could not move itself and the scow against the current. As soon as the spuds were lifted, the weight of the dipper on the bottom was not sufficient to hold the dredge, and it would slip back. The suggestion was made by W. J. Douglas, M. Am. Soc. C. E., Deputy Chief Engineer of the Canal Company, to turn the dredge at slack water, taking up a new position, so that it would always be working in the direction of the current. This movement was easily performed, the crew soon becoming so expert that substantially continuous service was secured.

The great obstacle to be overcome in the completion of the excavation was the removal of the boulders, which were found in surprisingly large numbers, even when the canal was considered finished. The small lumps which were expected to flatten out by wave action were found in nearly every case to be the ends of boulders, and to remove them meant the digging up of boulders that were embedded many feet. Similar experience was had on both banks, where the existence of boulders could be detected only by divers. Other boulders were found lying along the toes of the side slopes, whither they had been apparently rolled by the dipper-dredges.

These boulders were located by sweeping and by divers, and were removed either by hoisting in slings, attached by the divers, by large steam lighters, or were broken by blasting and the pieces disposed of. The blasting was done usually by placing charges of dynamite on top of the boulder, depending on the overlying water to act as tamping. The explosive used was du Pont 75% gelatine dynamite, the charge varying from 25 to 200 lb. The best results seemed to be produced by charges of about 50 lb., even if they had to be repeated, the second and subsequent charges being inserted in the fissures made by the first.

Boulders as large as 80 tons have been thus disposed of; the total number handled by divers, either by blasting or slings, was about 700, weighing in the aggregate perhaps 3 500 tons.

A large piece of work teaches lessons of two classes: successes and mistakes. The second class is quite as important as the first, if not more so. The lessons of the first class usually need no historian, for they always speak for themselves, whereas those of the second class are too frequently buried and lost sight of. The work of excavation of this canal, the largest single item in its construction, is not without lessons of the second class.

The first of these was the failure to see that in work of this character the steam shovel is superior to the dredge. The excavation contract being taken by a company whose great experience was in dredging naturally led them to use that method of attack, overlooking the wonderfully elastic capabilities of the American steam shovel.

In open water, dredging is the only method of excavating. As many units as desired can be used, and progress is not absolutely dependent on any particular one. In the case of a canal excavated through land, unless some means are devised for attack at intermediate points, advance depends entirely on the ability of the dredges at the two headings to maintain continuous performance. In practice this is very far from realization. Any accident, any stoppage for repairs, and the whole advance at that end ceases. By the very nature of the machine, there can be no reserve. Interest, hire of plant, overhead expenses, and much labor cost continue, regardless of whether progress is or is not made. Economy, therefore, demands that progress should be as nearly continuous as possible, in order to carry this heavy overburden. In the case of the Cape Cod Canal, the irregularity in the character and composition of the soil greatly hampered successful

dredging operations. A single boulder would frequently delay a whole unit—dredge, tugs, and scows—for many hours. Even when not delayed, progress was measured by the capacity of only two machines.

By the use of steam shovels, the reverse takes place. Each unit is comparatively inexpensive, and is independent of any other. Therefore, many can be used economically, and shovels can be held in reserve to take the place of any temporarily disabled, reducing loss through idle plant to the minimum. Thus a long stretch of work can be covered with plant, instead of having it concentrated at two points. With soil so variable in character as that at Cape Cod, it is of the greatest benefit to have it exposed to sight. Boulders beneath water were annoying and, through the delay to plant, expensive, but in the dry bothered steam shovels scarcely at all. If too large to lift, they were rolled to one side, and, after the shovel had passed, were broken by blasting and the parts picked up on the next cut.

Every one admitted the feasibility of removing by shovels the material lying above ground-water level, and the general contractors began such work as soon as it was seen that rapid progress by hydraulic dredges was not to be expected. The fear was of the difficulty of successfully and permanently lowering the water level. Experience showed that there was no serious difficulty in keeping the trench unwatered. The economical programme would have been to have constructed dams at about Station 112, where there was a highway, and at Station 317, where there was a railway embankment, and to have put in a battery of large steam shovels, with dippers of 5 cu. yd. capacity, served by standard gauge equipment. These shovels would have handled without trouble all the small boulders and fragments of the blasted ones, and the large cars would have received them without damage. Between these limits there were about 6 500 000 cu. yd., of which about 4 500 000 cu. yd. were below the level of ground-water. One-half of the canal proper (20 500 ft.) could have been thus unwatered and fully completed in the dry, with all boulders removed, slopes trimmed to even planes, floor leveled, and banks carefully protected by hand-placed rip-rap.

While this work was progressing the part east of Station 112 could have been excavated by a hydraulic dredge, the Scusset marshes affording convenient spoiling area, a few hard lumps being removed at the end by a dipper-dredge. West of Station 317 two dredges of the *Warfield* type should have been built, and should have worked in tandem,

completing the channel from Wings Neck Light. In the approach channel in Buzzards Bay there were about 3 250 000 cu. yd., and between the west end of the canal proper and Station 317 about 1 750 000 cu. yd. Of the latter perhaps one-half could have been put ashore by a light-draft hydraulic dredge, as was done in part, making a good channel for one of the larger dipper-dredges to work in completing to Station 317. In this way there would have been used two large dipper-dredges, one large hydraulic dredge, one small hydraulic dredge, one clam-shell to cut through the east beach, as was done by the *Nahant,* and four or five large steam shovels. Such a plant, by putting practically the whole length under construction simultaneously, could have completed the canal in 3 years. Instead of ten units there were actually used no less than twenty-six.

The second lesson was the use of antiquated instead of modern plant. In Table 1 the work of fifteen dredges is shown comparatively by giving the total output of each dredge while it was on the work, the equivalent number of months worked, and the average per month. This table includes all vicissitudes of the work—time lost through stress of weather, repairs, delay in the scow service, and other causes. The average, therefore, is one of good and bad, and the time each dredge worked was sufficiently long to give a fair average of all conditions. The number of months worked is a fair reduction of the time given in months when the dredge was actually in service, including the time laid up for small repairs. Where the dredge was absent from the work for one whole month that time has not been included. The names are not given, except in the cases of the *Warfield* and the *Herrick,* as it is unnecessary to make special and invidious comparisons.

TABLE 1.—COMPARISON OF THE WORK OF DREDGES.

Dredge.	Total output, in cubic yards.	Months.	Average per month, in cubic yards.
Warfield	1 265 800	14	90 000
Herrick	1 294 800	15.5	83 600
Dipper-dredge, No. 1	583 500	14	41 700
Ladder-dredge, No. 1	1 064 400	26	40 900
Dipper-dredge, No. 2	603 800	17	37 000
Dipper-dredge, No. 3	1 197 600	39.5	30 300
Dipper-dredge, No. 4	822 200	33	24 900
Dipper-dredge, No. 5	779 300	35	22 300
Seven other small dredges	732 800	48.5	15 000

From Table 1 it will be seen that the *Warfield* did nearly two and one-quarter times as much work as the nearest dipper-dredge, and four times as much as the poorest dipper-dredge, exclusive of the small dredges the output of which has been appended with an average of 15 000 cu. yd. per month. The *Warfield* was no more expensive to operate than Dipper-dredge No. 1, as the latter machine was very costly on account of repairs. The other machines were at least two-thirds as expensive to operate as the *Warfield,* although their output was very much less. In one month the thirteen other dredges would excavate, on the average, 302 000 cu. yd., or a trifle more than three times the average output of the *Warfield.* In this respect it must be kept in mind that the hard work was saved entirely for the *Warfield* and *Herrick,* as the other dredges were incapable of handling it.

It will be seen, therefore, that had specially designed dredges been set to work at the outset, they would have more than paid for themselves during the construction of the canal.

As an interesting comparison, the two 20-in. hydraulic dredges removed 2 558 300 cu. yd. in 26 months, or an average of 100 000 cu. yd. per month, and the small hydraulic dredge *Federal* removed 613 800 in 32 months, or an average of 19 200 cu. yd. per month. The excavators removed 316 200 cu. yd. in 22½ months, an average of 14 000 cu. yd. per machine per month; and the steam shovels removed 2 077 600 cu. yd. in 89½ months, an average of 23 200 cu. yd. per shovel per month, and these were small shovels served by narrow-gauge equipment, put in as an experiment. The excavators and the shovels worked on a single shift, but all the dredges worked continuously on double shift.

Including excavation made by the Metropolitan and Standard dredges and by other dredges used directly by the Canal Company, the total excavations amounted to about 15 000 000 cu. yd.

Breakwater.—The breakwater at the east end was necessary to protect the mouth of the canal from littoral drift and to afford protection to vessels entering and leaving in time of heavy weather.

The principal clauses in the specifications describing it are as follows:

The shore end shall extend from the mean high water mark at a uniform height of 10 ft. above the beach until it intersects the bluff or dune. The beach and the sides of the dune forming the surface of

this intersection shall be rip-rapped for the distance and to the extent as directed to form a secure revetment, with stones weighing at least 100 lb.

The stone will be deposited in the structure so as to construct a breakwater of a uniform width of 25 ft. at a point 18 ft. above mean low water.

From the top to a point 12 ft. below mean low water the seaward or northeasterly slope shall be 1 on 2, and thence to the base the slope on the seaward side shall be 1 on 1.

The harbor or southeasterly side, shall be constructed throughout with a slope of 1 on 1.

The mean rise and fall of the tide is about 10 ft.

The stone must weigh at least 160 lb. per cu. ft., must be strong and durable, and not subject to disintegration by being wholly or partly submerged in sea water.

Below the level of 12 ft. below mean low water, and in the core, that is to say, not nearer than 10 ft. from the outside line of the breakwater, the stone may be of any size convenient, provided that no stone shall be of less weight than 100 lb., and that, at least 50% of the stones shall weigh not less than 1 ton each.

Within the space above defined, *i. e.,* the core below 12 ft. below mean low water, coarse gravel and sand shall be dumped into and among the stones as directed, and to the quantity ordered by the Engineer.

Below 12 ft. below mean low water and on the faces of the breakwater, *i. e.,* the outward 10 ft. of each slope, no stone shall be less than 500 lb. in weight and at least 50% shall be 2 tons or more each in weight.

Above the level of 12 ft. below low water no stones shall be used that weigh less than 3 tons each, except that where directed smaller stones may be used to fill in openings and to provide firm bearings for the larger stones, and at least 25% must weigh 6 tons each. Nor shall any stone be used in this position of which the least dimension is less than one-fourth of its greatest.

In the absence of scales or other convenient method of weighing separate stones, the judgment and decision of the Engineer as to the weight of various stones and as to the proportions of stones of various weights as specified in the above paragraphs shall be final and binding upon the Contractor.

The weight of stone deposited will be determined by water displacement, and in order to determine the correct displacements, the Contractor may be required to have the vessels accurately "weighed in" and distinctly marked, at his own expense; the "weighing in" and marking to be done under supervision of the Engineer. In "weighing in" the vessel must be kept in the same fore and aft trim

that is to be used when freighting stone. When fore and aft readings of a loaded vessel differ by more than 10% of their mean, the Contractor will be required to move enough stones to make the difference between the fore and aft readings less than 10% before the stone will be received.

As the work progressed it was decided by the Engineer to depart from the usually accepted standard of breakwater construction, as illustrated in the foregoing extracts from the specifications, and omit any attempt at compact hearting by using only such small stones as came normally mixed with the large ones, and omitting all gravel and sand. The result is a breakwater with large voids, such as naturally form between large irregular blocks. The resulting effect seems to be that any vacuum following a receding wave is impossible, the large voids giving free movement to the air. Waves on striking the breakwater are broken partly by direct shock and partly by dissipation in these large voids. No stones have been displaced on the seaward side since the final setting, except at the extreme outer end, as will be referred to presently; and, in spite of the large voids, sand does not apparently travel through the breakwater. On the shore end the beach has built itself out so as to make a curve between the face of the breakwater and the shore line, which are at right angles to each other. Where the sand is thus built up it is 10 ft. higher on the outside than on the inside of the breakwater, which at that point has a thickness of 62 ft. The sand, therefore, seems to assume a slope through the breakwater of about 1 on 6.

Fig. 4 shows a cross-section of the breakwater as originally planned. In addition to the change in specifications describing composition, one change was made in the profile during construction. Although the stones were stable at a slope of 1 on 1 on the inside face, the contractors found difficulty in placing them at that slope, so that they were permitted to flatten it from low-water level up by narrowing the top from 25 ft., as shown, to about 22 ft.

The total length of the breakwater is 3 000 ft., carrying the outer end to where the water is 35 ft. deep at mean low water, the total height of the breakwater at the end being, therefore, 53 ft. In its construction, 326 456 tons of stone were used.

Parallel with the breakwater and 800 ft. from it, a smaller breakwater has been built on the south side of the channel. This smaller

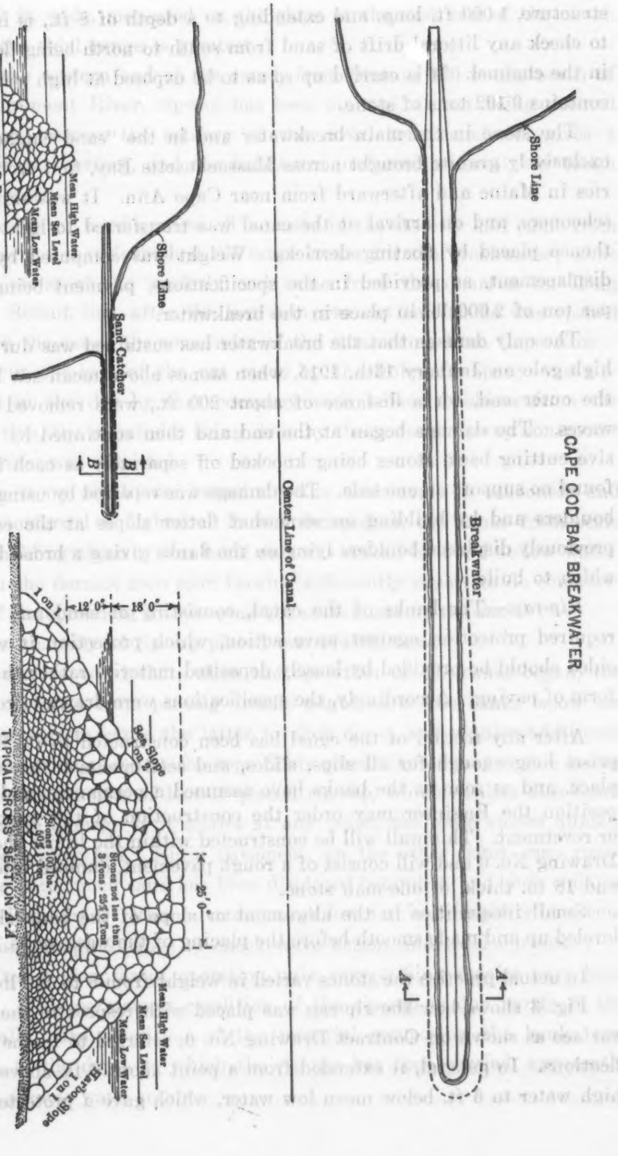

CAPE COD BAY BREAKWATER
Shore Line
Breakwater
Center Line of Canal
Shore Line
Sand Catcher
B
B
A
A
SECTION B-B
Mean High Water
Mean Sea Level
Mean Low Water
TYPICAL CROSS SECTION A-A
12'0"
18'0"
25'0"
1 on 2
Sea Slope
Mean High Water
Mean Sea Level
Mean Low Water
Harbor Slope

FIG. 4.

structure, 1 000 ft. long, and extending to a depth of 8 ft., is intended to check any littoral drift of sand from south to north being deposited in the channel. It is carried up so as to be exposed at high water, and contains 9 192 tons of stone.

The stone in the main breakwater and in the "sand catcher" was exclusively granite brought across Massachusetts Bay, first from quarries in Maine and afterward from near Cape Ann. It was shipped in schooners, and on arrival at the canal was transferred to lighters and thence placed by floating derricks. Weight was computed by vessel displacement, as provided in the specifications, payment being made per ton of 2 000 lb. in place in the breakwater.

The only damage that the breakwater has sustained was during the high gale on January 13th, 1915, when stones above mean sea level at the outer end, for a distance of about 200 ft., were removed by the waves. The damage began at the end and then continued by successive cutting back, stones being knocked off separately as each in turn found no support on one side. The damage was repaired by using larger boulders and by building on somewhat flatter slopes at the end, the previously displaced boulders lying on the flanks giving a broad base on which to build.

Rip-rap.—The banks of the canal, consisting of sand and gravel, required protection against wave action, which protection it was decided should be provided by loosely deposited material rather than any form of paving. Accordingly, the specifications were drawn to read:

After any section of the canal has been constructed to its finished prism long enough for all slips, slides, and settlements to have taken place, and as soon as the banks have assumed a permanent and stable position the Engineer may order the construction of the wash walls or revetment. This wall will be constructed within the limits shown in Drawing No. 6 and will consist of a rough pavement, dumped into place and 18 in. thick, of one-man stone.

Small inequalities in the alignment or slope of the bank shall be leveled up and made smooth before the placing of the wash walls.

In actual practice the stones varied in weight from 5 to 150 lb. each.

Fig. 3 shows how the rip-rap was placed with respect to the water surface as shown on Contract Drawing No. 6, referred to in the specifications. In general, it extended from a point about 6 ft. above mean high water to 6 ft. below mean low water, which gave a protected sur-

face 42 ft. wide, measured on the slope at the east end of the canal, where the tidal range is greatest.

At the western end of the canal, from Station 360 to the north of the Monument River, rip-rap has been omitted, as not being deemed necessary because the estuary of the river, here canalized, gives a wider water surface, and the banks are protected naturally by tough marsh grass.

The rip-rap is of granite. That used in the easterly half came from the quarries that furnished the large stones for the breakwater; some of the stone at the westerly end came from the quarries on Long Island Sound, but, after the last dam was cut and traffic was opened through the canal, the remainder of the stone came from Cape Ann.

About one-half of the cut made by the Foley Company was rip-rapped in the dry by that company with broken boulders from the excavation, the remainder being done by the general rip-rap contractor after water was admitted.

The stone which was brought to the work came in schooners and was transferred to lighters. It was dumped on the banks from scale boxes holding about 2 tons each, and was spread by hand at low water, though the derrick men soon became sufficiently expert to do considerable of the spreading by sweeping the scale boxes up the back.

The quantity of rip-rap placed was 144 397 tons.

It was expected that, when the operation of the canal began, the effect of waves from passing vessels might scour the banks below the rip-rap so as to cause the latter to slide down, and require additional material to be added to the top. This possibility of easy repair is one of the great merits of loosely placed rip-rap, which quality also tends to localize damage if it settles at any point, the loose stones quickly finding a new bed. Rigid pavement, on the other hand, may resist falling until much sand has been displaced behind it, and then collapse over a large area. Other advantages are low first cost, rapidity of execution, and rough surface to check wave action and tidal current.

After 30 months of operation only very trifling repairs have been made. This permanent condition of the rip-rap is due partly to the naturally hard condition of the material composing the banks and partly to the depth to which the rip-rap has been carried, apparently below serious wave action.

Work on the breakwater was begun in June, 1909, and was finished in December, 1913, in 35 months of actual working time. It could have been completed earlier had there been necessity. Progress in placing rip-rap was naturally governed by canal excavation, but was at the maximum in 1914, when 8 400 tons were placed in 1 month.

Railway Reconstruction.—The Old Colony Railroad Company, now leased to the New York, New Haven and Hartford Railroad Company, owned the line from Boston to the Cape. It was a double-track line from Boston to the station known as Buzzards Bay, where it forked into two single-track lines, one crossing the Monument River at Buzzards Bay and running to Woods Hole, the other turning east and running up the valley of the river and thence along the hook of the cape to Provincetown. The latter line not only occupied a portion of the lands required for the canal location, but actually crossed the canal line three times.

The Canal Company's charter contemplated the taking of the Railroad Company's right of way, under proper safeguards to the latter's operation. Conferences with the engineers of the New Haven and the Old Colony Railroad Companies—for the latter still maintains its full corporate existence—resulted in an agreement:

To relocate both branches from Buzzards Bay station, and, though leaving the actual physical junction at the station, to make a single crossing of the canal for both lines, with the point of divergence on the south bank;

To relocate a short piece of the Woods Hole line from the point of divergence to connect with the existing line;

To relocate the Provincetown line along and near the south side of the canal right of way until it intersected the old line east of Bourne, and again to relocate a piece at Bournedale, where the railroad, in order to avoid heavy cutting, crossed and recrossed the canal location.

Previous to these changes, the junction of the lines was at the north end of the Buzzards Bay station, but after the relocation was completed the junction point was at the south end of the station, and this change required a complete reconstruction of the Buzzards Bay yard. The old junction and yard switches, with their signals, were controlled by lever operation, a system that could not be altered to in-

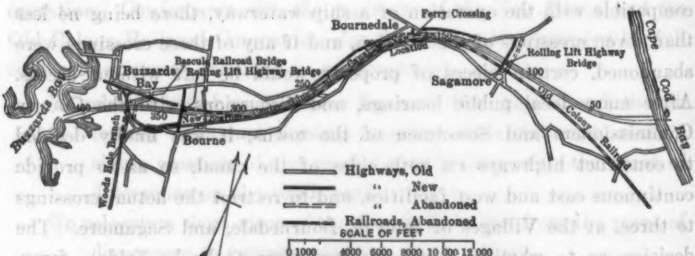
clude a draw-bridge with its home and distant signals on the far side of a waterway all interlocked and controlled from a central tower. The mechanical system, therefore, was abandoned and a new electro-pneumatic plant put in. All told, there were 6.3 miles of single main track and 1.2 miles of side track newly constructed. The quantity of track work was not large, but the expense was proportionately high, as the yard, signals, water supply, and bridges brought the total cost to $379 274.05, in addition to which the Railroad Company bore the expense of a new station building and certain extensions of the signal system that were not deemed to be called for by the presence of the canal. The only item in the railroad construction that is of interest is the drawbridge, which, with the highway bridges, is described in detail. The contract for railroad work was taken by the Degnon Contracting Company and sublet to the Wilson and English Construction Company.

Highways.—The highways proved an annoying detail to adjust. Those existing before the canal construction began were wholly incompatible with the operation of a ship waterway, there being no less than seven crossings of the location, and if any of these crossings were abandoned, certain pieces of property would be left without access. After many local public hearings, and discussions with the County Commissioners and Selectmen of the towns, it was finally decided to construct highways on both sides of the canal, so as to provide continuous east and west facilities, and to restrict the actual crossings to three, at the Villages of Bourne, Bournedale, and Sagamore. The decision as to whether these crossings were to be by bridge, ferry, or tunnel was by the charter left to the Joint Board, who ordered bridges at Bourne and Sagamore, and a ferry for passengers only at Bournedale. These bridges will be considered in connection with the railroad bridge.

New highways, 4.4 miles in length, including the portions occupied by bridges, were constructed, and 0.6 mile was resurfaced, all of which the local authorities insisted should be built to a standard never before observed in that part of Massachusetts, involving heavy earth cuttings and embankments.

Fig. 5 shows the railroad and highways as they existed before and after relocation, with the points of crossing.

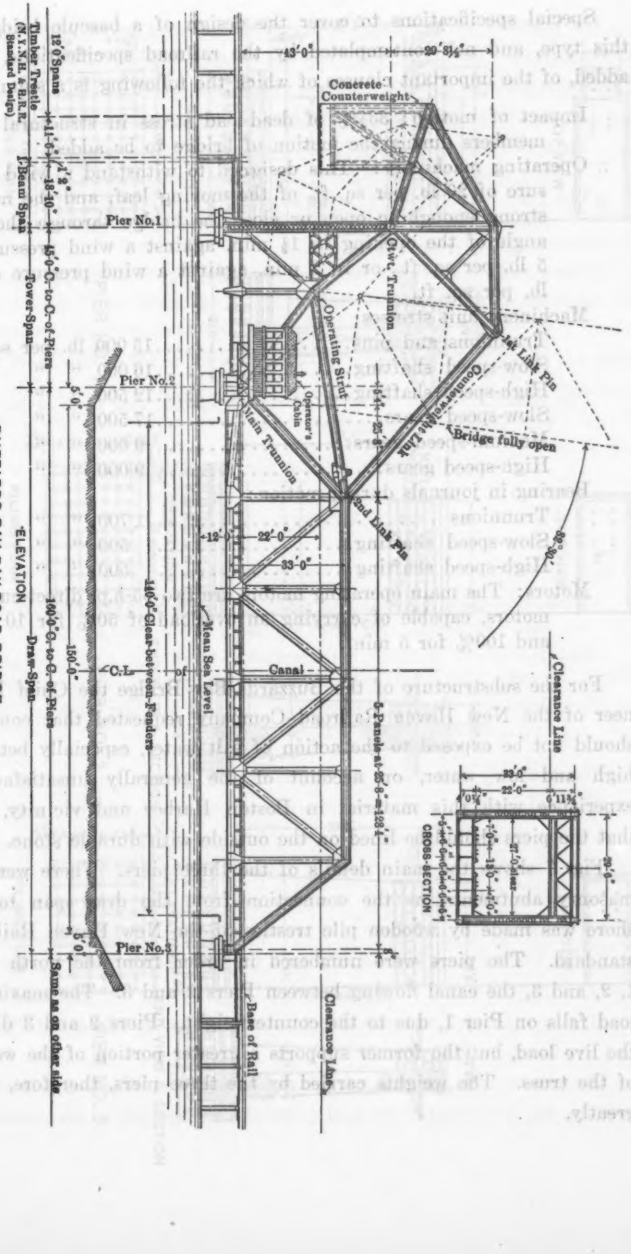
Bridges.—There are three bridges across the canal. The charter required the canal to have a prism with a bottom width of 100 ft., but the directors realized that it would be only a question of a short time when a wider canal would be required—one sufficiently broad to permit tows to pass in opposite directions, and a deeper canal to accommodate battleships or vessels drawing more than 25 ft. Although the canal can be deepened or widened at any time, bridges are rigid obstructions that can be altered only by complete reconstruction. The directors, therefore, decided to build in the first instance bridges larger than were demanded to meet the requirements of the charter, and large enough to fit a canal of such increased dimensions as could be reasonably foreseen as probably necessary. After considering various designs and estimates, channel draw-spans of 160 ft. in length, from center to center of piers, were adopted, with foundations to a depth permitting the canal to be made more than 30 ft. deep.



RAILROAD AND HIGHWAY RELOCATION

FIG. 5.

Swing bridges were out of the question, as central piers would completely block a straight channel. For the railroad bridge, after conferences with the engineers of the New Haven Railroad Company, the Strauss type of trunnion bascule bridge was selected, to be built to meet the New York, New Haven and Hartford Railroad bridge specifications of 1908, which contemplate two engines of consolidation type with 60 000 lb. per driving axle, 30 000 lb. leading axle, and 39 000 lb. per tender axle, or 65 000 lb. per axle on two axles 7 ft. apart for details. The general elevations and cross-section are shown on Fig. 6.



BUZZARDS BAY RAILROAD BRIDGE

FIG. 6.

Special specifications to cover the design of a bascule bridge of this type, and not contemplated by the railroad specifications, were added, of the important clauses of which the following is a summary:

Impact of motion: $33\frac{1}{3}\%$ of dead-load stress in structural steel members during the motion of bridge to be added.

Operating machinery: This designed to withstand a wind pressure of 20 lb. per sq. ft. of the moving leaf, and the motors strong enough to open or close the bridge through the full angle of the opening in $1\frac{1}{2}$ min. against a wind pressure of 5 lb. per sq. ft., or in 2 min. against a wind pressure of 15 lb. per sq. ft.

Machinery unit stresses:

Trunnions and pins	15 000	lb.	per sq.	in.
Slow-speed shafting	16 000	"	"	"
High-speed shafting	12 500	"	"	"
Slow-speed gears	17 500	"	"	"
Medium-speed gears	9 000	"	"	"
High-speed gears	9 000	"	"	"

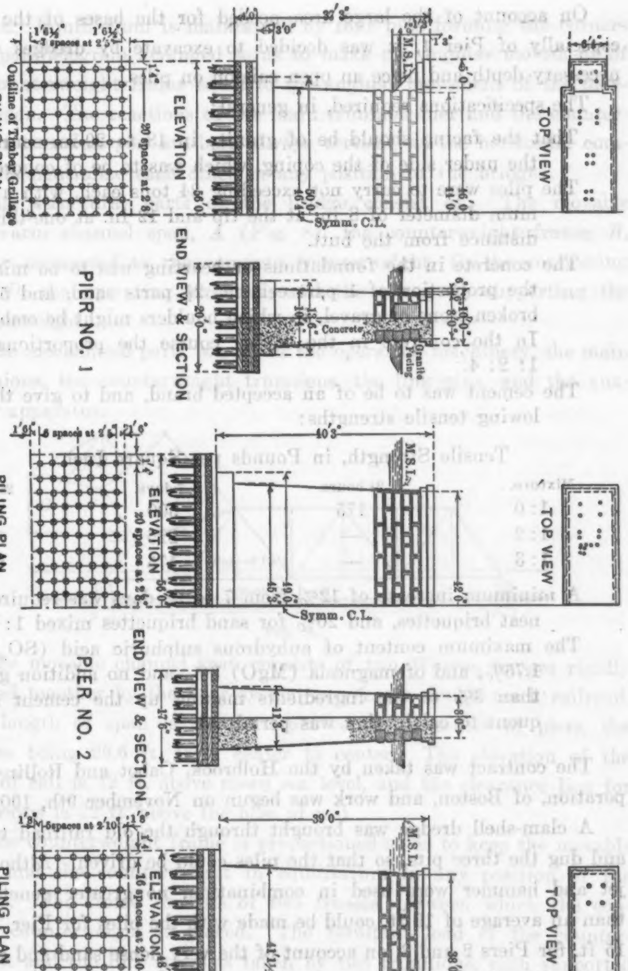
Bearing in journals during motion:

Trunnions	1 700	"	"	"
Slow-speed shafting	500	"	"	"
High-speed shafting	500	"	"	"

Motors: The main operating motors are two 65-h.p. direct-current motors, capable of carrying an overload of 50% for 10 min. and 100% for 5 min.

For the substructure of the Buzzards Bay Bridge the Chief Engineer of the New Haven Railroad Company requested that concrete should not be exposed to the action of salt water, especially between high and low water, on account of the generally unsatisfactory experience with this material in Boston Harbor and vicinity, and that the piers should be lined on the outside with durable stone.

Fig. 7 shows the main details of the three piers. There were no masonry abutments, as the connection from the draw-span to the shore was made by wooden pile trestles of the New Haven Railroad standard. The piers were numbered in order from the north end: 1, 2, and 3, the canal flowing between Piers 2 and 3. The maximum load falls on Pier 1, due to the counterweight. Piers 2 and 3 divide the live load, but the former supports a greater portion of the weight of the truss. The weights carried by the three piers, therefore, vary greatly.



PIERS FOR BUZZARDS BAY RAILROAD BRIDGE

FIG. 7.

On account of the large area needed for the bases of the piers, especially of Pier 1, it was decided to excavate by dredges to the necessary depth and place an open caisson on piles.

The specifications required, in general:

That the facing should be of granite in 18 to 20-in. courses, to the under side of the coping, which was to be of concrete.

The piles were to carry not exceeding 24 tons each, with a minimum diameter of 8 in. at the tip and 12 in. at one-third the distance from the butt.

The concrete in the foundations or hearting was to be mixed in the proportion of 1 part cement, 2½ parts sand, and 5 parts broken stone or gravel, in which boulders might be embedded. In the concrete in the coping course the proportions were 1: 2: 4.

The cement was to be of an accepted brand, and to give the following tensile strengths:

Tensile Strength, in Pounds per Square Inch.

Mixture.	24 hours.	7 days.	28 days.
1:0	175	500	575
1:2	—	225	300
1:3	—	175	225

A minimum increase of 12% from 7 to 28 days was required for neat briquettes, and 20% for sand briquettes mixed 1: 2.

The maximum content of anhydrous sulphuric acid (SO_3) was 1.75%, and of magnesia (MgO) 3%, and no addition greater than 3% to the ingredients making up the cement subsequent to calcination was permitted.

The contract was taken by the Holbrook, Cabot and Rollins Corporation, of Boston, and work was begun on November 9th, 1909.

A clam-shell dredge was brought through the old railroad trestle, and dug the three pits, so that the piles could be driven. Although a jet and hammer were used in combination, no greater penetration than an average of 19 ft. could be made with the piles for Pier 1, and 15 ft. for Piers 2 and 3, on account of the very dense sand and gravel. The specifications called for 20 ft., and, to make up for the deficiency, twenty-two extra piles were driven under Pier 2. The details are clearly shown on Fig. 7.

The elementary features of the superstructure, which is a Strauss bascule, are fixed trunnions and parallel link motion of the counter-

weight. Equilibrium is maintained by four pins forming the corners of a parallelogram arranged so as to make the angular movement of the counterweight frame equal to the angular movement of the movable span. The reactions of the main trunnion pier and the counterweight trunnion pier are both always vertical, as the horizontal components neutralize each other in any position of the bridge.

The structural parts of the bridge consist of: The movable single-arm channel span, *A* (Fig. 8), the counterweight frame, *B*, rigidly connected to the concrete counterweight, *C*, the connecting link, *D*, the operating strut, *E*, and the tower, *F*, supporting the counterweight.

The mechanical parts consist of the operating machinery, the main trunnions, the counterweight trunnions, the link pins, and the auxiliary apparatus.

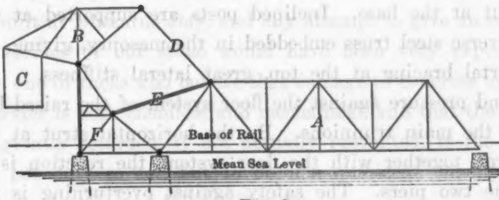


FIG. 8.

The movable channel span consists of two Warren trusses rigidly framed together by the floor system carrying a double-track railroad. The length of span is 160 ft. from center to center of piers, the trusses being 29.6 ft. from center to center. The elevation of the base of rail is 12 ft. above mean sea level, and the clearance line for the trains is 22 ft. above the base of rail.

The counterweight frame is proportioned so as to keep the movable span and the counterweight in equilibrium in any position of the bridge. The frame consists of two trusses, between which the concrete counterweight is placed. The resultant load of the counterweight and the link stress is taken by two trunnions, each supported by two bearings at the apex of the tower. Special collars riveted to the frame are bored with a driving fit, and the trusses are fastened to the trunnion with tapered keys. The counterweight is attached to the rear arm of the truss, and rigidity is attained by extending the structural framing into the concrete. Besides the weight of

the concrete, pockets are provided for shaped iron castings, which allow for close adjustment in balancing the bridge. The link strut consists of two posts, pin-connected at the top to the front arm of the counterweight frame, and at the bottom to the hip of the movable span. Lateral stiffness is secured by diagonal and horizontal bracing. The link, at any angle of the movable leaf, is always parallel to a line which travels through the center of the main and counterweight trunnion.

The operating struts to which the main rack is attached are pin-connected at one end to the tower, and at the other are engaged to the main operating pinion. A special yoke, mounted on each side of the main pinion, serves as a guide for the strut during operation.

The tower supporting the counterweight consists of two main columns, a diagonal strut between the two trunnions and a horizontal strut at the base. Inclined posts are supported at the base on a transverse steel truss embedded in the masonry, giving, in addition to portal bracing at the top, great lateral stiffness.

The wind pressure against the floor system of the raised bridge is carried to the main trunnions. By the horizontal strut at the base of the tower, together with the floor system, the reaction is divided between the two piers. The safety against overturning is thus increased, and the corresponding eccentric loading on the foundations is materially reduced. All trunnions and link pins are of forged steel. The bearings and caps are of cast steel and lined with phosphor-bronze bushings. Helical oil grooves are cut in the caps and bearings for the proper distribution of the lubricants.

The motive power for operating the bridge is electricity. Two 65-h.p. electric motors operate under a 550-volt direct current. The power is transmitted to the main operating pinion by a train of gears. In addition to the electric brake on the motor, there is also a motor-driven emergency brake. Special equalizing gears regulate the uniformity of speed between the duplicate system of operating machinery. The motors and gearing are on the movable span, and are protected by special housing. A separate motor with the necessary mechanical connections operates the locking device.

All machinery and auxiliary apparatus are operated by controllers, switches, switch-boards, indicators, and instruments conveniently placed in the operator's house. From this point the operator has per-

fect control of all traffic passing through the canal and over the bridge. The control of the bridge is interlocked with the main tower controlling all signals and switches in the Buzzards Bay yard, although arranged so that the tower operator can free the bridge for independent operation.

The highway bridges at Bourne and Sagamore are of the double bascule type, of the same dimensions and almost identical design. It will be convenient, therefore, to treat the substructures and superstructures of these bridges together.

The substructure for the highway bridge at Bourne was designed on quite different lines from that of the railroad bridge at Buzzards Bay, a design that was followed with few changes in details for the highway bridge at Sagamore. On account of the great lightness of the highway bridge, there was no need for such a massive substructure as was built at Buzzards Bay, and any attempt to give an exterior protective surface of cut stone would have been very expensive. The writer is one of those who believe that damage to concrete by the action of salt water is both chemical and mechanical, and that the destruction can be greatly retarded, if not actually prevented, if the concrete is allowed not only to set, but to become quiescent chemically, before exposure to the rise and fall of the tides.

The most economical type of substructure was an isolated support beneath each point of load, varying in diameter according to the load intensity. Such a type was selected, and in order to keep the surface of the exposed piers from tidal exposure until internal changes had ceased, the specifications required that the forms for the concrete should be of surfaced lumber and left in place. As the teredo is very active, creosoted lumber was called for. The intention was to let the protective wood remain until it was destroyed by natural agencies, or for at least a year, and then to take it down. As a matter of fact, it is still in place and in good condition, both at Bourne and Sagamore. The lumber being matched and not unsightly, it is not necessary for appearance to remove it. The concrete is now more than 5 years old, and, as shown by recent borings through the lumber, is in perfect condition. The writer believes that, if the lumber should be removed, no injurious effect would now result from the action of salt water during either the summer or winter.

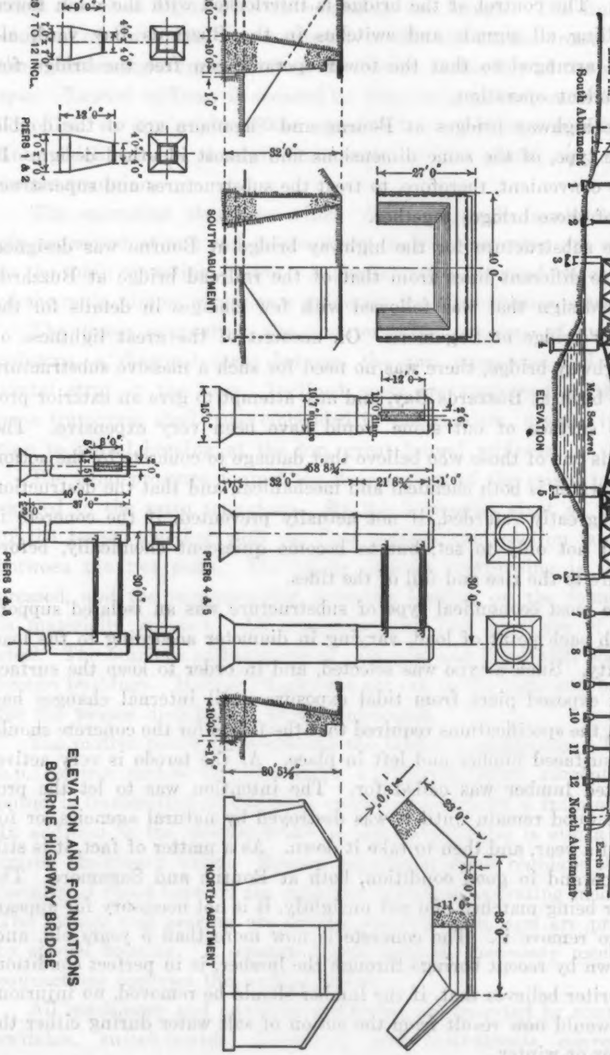


FIG. 9.

ELEVATION AND FOUNDATIONS
BOURNE HIGHWAY BRIDGE

The specifications were similar to those for the foundations of the Buzzards Bay Bridge, except that no piles were contemplated, and pressure on the sand was limited to 3 tons per sq. ft. The requirements for cement and concrete were the same, a mixture of the latter in the proportion of 1:2:4 being used in the upper 2 ft. of the main and secondary piers.

Fig. 9 shows the general elevation of the Bourne highway bridge and details of the substructure. The main piers, Nos. 4 and 5, and the secondary piers at the ends of the approach girders, are square in section, 15 ft. 1 in. and 8 ft. 10½ in. on bases, respectively, and 10 ft. 7 in. and 5 ft. 4 in. on the shafts to Elevation 96, and above that elevation, circular. In order to stiffen these piers laterally, reinforced concrete girders were designed for all piers, these girders being 12 ft. deep and 2 ft. 6 in. wide in the case of the main piers, and 8 ft. deep and 2 ft. wide for the secondary piers.

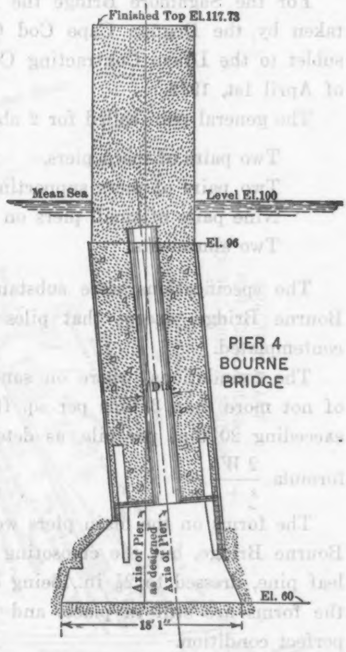


Fig. 10.

All the caissons for these eight piers were sunk by compressed air and then filled with concrete, the material through which they passed being for the most part fine sand. Pier 4 (the westerly one), just before reaching grade, was badly deflected by striking a boulder. One of the cutting sides of the caisson was cut away, and, under cover of poling boards, the sand was excavated unsymmetrically, as shown in Fig. 10, so as to bring the axis of pressure well within the middle third of the base, the increased area of base reducing the pressure at the outer edge to proper limits. Above Elevation 96 the pier was carried straight up. The heavy reinforced girder connecting the tops of the two main

piers affords sufficient rigidity to overcome the objection to the inclination.

The work was executed by the Holbrook, Cabot and Rollins Corporation, the layout of the erecting plant being shown by Fig. 11. Access to both sides of the river, the canal not being dredged at the time, was had by a temporary trestle, carrying a traveler and also a concrete car. A second traveler on the ground covered the piers at the north end.

For the Sagamore Bridge the contract for the substructure was taken by the Degnon Cape Cod Canal Construction Company and sublet to the Dravo Contracting Company of Pittsburgh, under date of April 1st, 1912.

The general plan called for 2 abutments and 26 piers:

Two pairs of main piers,
Two pairs of piers supporting approach girders,
Nine pairs of small piers on land,
Two abutments.

The specifications were substantially the same as those for the Bourne Bridge, except that piles beneath some of the piers were contemplated.

The foundations, where on sand or gravel, were to carry a load of not more than 3 tons per sq. ft., and, where on piles, a load not exceeding 20 tons per pile, as determined by the *Engineering News* formula, $\frac{2\,Wh}{s+1}$.

The forms on the main piers were to be left in place, as with the Bourne Bridge, but the creosoting of the lumber was omitted, long-leaf pine, dressed to 2⅝ in., being called for. At the end of 3 years the forms are still in place, and the concrete, as at Bourne, is in perfect condition.

For the caissons for the pair of main piers, No. 10, the contractor designed a steel shell of ⅜-in. plates, with the lower part 16 ft. 6 in. in diameter for a height of 10 ft., then reducing through a height of 4 ft. to a diameter of 12 ft. 5½ in. for a height of 36 ft., and there the wooden form began with an internal diameter of 10 ft. The shell weighed about 16 tons, and was sunk by removing the material from the inside with an orange-peel bucket. At Elevation 56.5 the

bottom was sealed by divers with concrete in bags to a height of 5 ft. Then 12 ft. of concrete were deposited under water in buckets, and after this had set the water in the shell was pumped out and the remaining concrete was placed in the dry.

Main piers No. 9 were sunk by the pneumatic process. The steel shell at the bottom was splayed at the foot to a diameter of 12 ft. … in. and lined with concrete, as shown in Fig. 12. The cutting edge was carried down to a gravel bed at Elevation …, and the concrete was carried down 0.4 ft. lower.

For Piers Nos. 8 and 11, supporting the ends of the 90-ft. approach girders, steel shells, 6 ft. 8 in. in diameter, were sunk by removing the material within the shells with an orange-peel bucket. In Piers No. 8 six 25-ft. piles were driven, reaching below Elevation 72, with concrete beginning at Elevation 9…. The similar piers on the other side of the canal were sent down in the same manner, except that concrete … vation ….

The abutments were founded directly on hard sand, but the abutment in the small pedestals, 1, 2, 3, and 4, rest on piles cut off below ground water level as required by the owners of the manufacturing plant on whose property they were placed, as they … damage arising from unknown presence of quicksand.

The main piers, Nos. 9 and 10, and the second piers, Nos. 8 and 11, are connected by reinforced concrete girders, as at Bourne.

The Sagamore Highway Bridge crosses the waterway about 2 miles west of the easterly entrance to the canal, and, as a structure, is divided into three sections: the main channel span, the north, and the south approach.

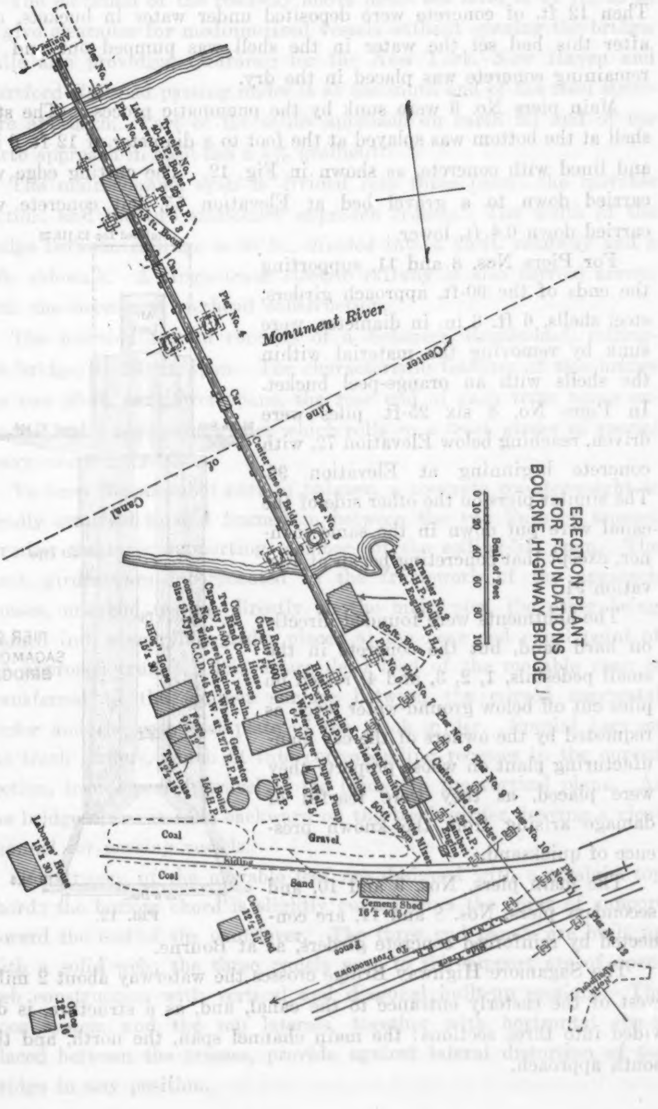


FIG. 11.

bottom was sealed by divers with concrete in bags to a height of 5 ft. Then 12 ft. of concrete were deposited under water in buckets, and after this had set the water in the shell was pumped out and the remaining concrete was placed in the dry.

Main piers No. 9 were sunk by the pneumatic process. The steel shell at the bottom was splayed at the foot to a diameter of 12 ft. 6 in., and lined with concrete, as shown in Fig. 12. The cutting edge was carried down to a gravel bed at Elevation 55, but concrete was carried down 0.4 ft. lower.

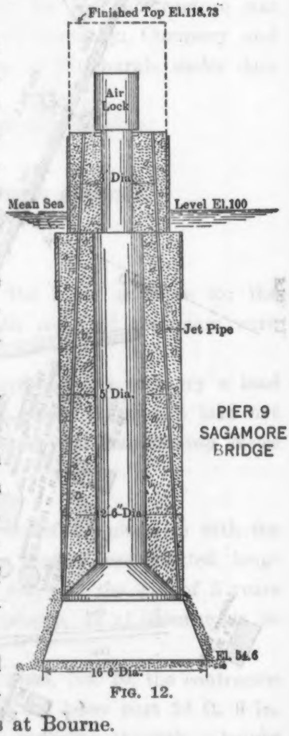


Fig. 12.

For Piers Nos. 8 and 11, supporting the ends of the 90-ft. approach girders, steel shells, 6 ft. 6 in. in diameter, were sunk by removing the material within the shells with an orange-peel bucket. In Piers No. 8 six 25-ft. piles were driven, reaching below Elevation 72, with concrete beginning at Elevation 92. The similar piers on the other side of the canal were put down in the same manner, except that concreting began at Elevation 91.

The abutments were founded directly on hard sand, but the concrete in the small pedestals, 1, 2, 3, and 4, rested on piles cut off below ground-water level, as requested by the owners of the car manufacturing plant on whose property they were placed, as they were fearful of damage arising from the known presence of quicksand.

The main piers, Nos. 9 and 10, and secondary piers, Nos. 8 and 11, are connected by reinforced concrete girders, as at Bourne.

The Sagamore Highway Bridge crosses the waterway about 2 miles west of the easterly entrance to the canal, and, as a structure, is divided into three sections: the main channel span, the north, and the south approach.

The elevation of the roadway above mean sea level is 41 ft., so as to give clearance for medium-sized vessels without opening the bridge, while also providing clearance for the New York, New Haven and Hartford Railroad passing under it at the south end of the steel structure approach. Part of the south approach on earth fill and of the north approach in steel has a 5% gradient.

The main-channel span is divided into three parts: the movable section, and the two stationary approach trusses. The width of the bridge between railings is 30 ft., divided into a 25-ft. roadway and a 5-ft. sidewalk. A single-track electric railway is also carried across, with the necessary overhead construction.

The movable section consists of a Scherzer, double-leaf, rolling-lift bridge, of 160 ft. span. The characteristic features of this bridge are two 80-ft. cantilever spans, the rear end of each truss being extended into a segmental girder which rolls on a track girder of special heavy construction.

To keep the movable part in balance, a concrete counterweight is rigidly attached to and framed in between the two lines of trusses, forming the main supporting members of the rolling-lift span. The track girders are incorporated in the framework of the approach trusses, one end resting directly on the main pier, the other being framed into a carrying girder placed at the near end panel point of the approach trusses. The entire dead load of the movable span is transferred to the contact surface between the curved segmental girder and the cast-steel plate of the track girder. Special lugs on the track girders, which fit into corresponding recesses in the curved section, insure perfect travel of the bridge in the vertical plane. As the bridge opens it rolls backward on the track girder, leaving a clear channel for passing vessels.

The trusses of the movable leaf are designed with a straight top chord; the bottom chord is slightly curved from the point of support toward the end of the cantilever. The three end panels are built up with a solid web; the three panels next to the support are of open-web construction with vertical and diagonal built-up sections. The floor system and the top laterals, together with horizontal struts placed between the trusses, provide against lateral distortion of the bridge in any position.

The approach trusses, each of 90-ft. span, are of the Warren type with stiff riveted web members. The trusses are spaced so as to provide proper clearance for the receding counterweight as the bridge opens. Any danger of damage resulting from the upward reacting force at the rear end of the lift span, due to unbalanced live load or impact in closing, is guarded against by a bumping girder placed on the top chord of the approach span. The ends of each movable leaf are provided with a shear lock designed with male and female parts, which in a closed position are interlocked, so as to insure proper alignment and additional lateral stiffness.

The north and south approach spans are in lengths of 38, 50, and 75 ft. The approach spans are of conventional design, with the two main girders seated on top of the columns, laterally connected by horizontal struts and diagonal bracing. The floor-beams throughout the entire length of the bridge are about 13 ft. apart, and are framed into the web of the main trusses or girders. On the channel approach trusses, they rest on the top chord. The roadway stringers consist of 9-in. I-beams about 2 ft. 6 in. from center to center, except for the support of the railway track, where 10-in. I-beams are placed in pairs under each rail. Bolted to the top flange of each stringer are nailing strips, to which is fastened the 4-in. oak plank decking, being laid with $\frac{1}{4}$-in. joints and sufficiently crowned to provide for quick run-off of rain water. The sidewalk brackets are in line with the floor-beams, and carry wooden stringers to which the 2-in. plank flooring is fastened. On the lift span there are special fastenings to guard against possible movement of the floor system when the bridge is being raised. The hand-railing is of 2-in. gas pipe posts, from 6 to 7 ft. apart, and joined by two lines of $1\frac{1}{2}$-in. pipes at the top and near the base of the posts, the panel between the two lines of pipe consisting of a grillwork of light angles and bars.

The motive power for operating the bridge is electricity, furnished at 550 volts, each movable leaf being operated by a 25-h.p. direct-current motor. The power transmission from the motor to the main operating pinion is arranged by gear trains and countershafts mounted on the movable span. The horizontal rack which engages the main pinion is attached to the approach trusses with the pitch line parallel to the horizontal top flange of the track girder.

There are safety gates on the approach trusses directly in front of the break in the floor system; they are opened and closed from the operator's house. From this point the bridge tender has under his control all the electrical apparatus for the operation of the machinery, signals, lights, and auxiliary mechanical devices for the protection of the structure itself, as well as the regulation of traffic over the bridge and through the canal.

The Bourne Highway Bridge, which crosses the canal about 1½ miles east of the westerly entrance, is designed along the same lines as the Sagamore Bridge, the only difference between the two structures being the length of the approaches and the width of the roadway, the latter, for the Bourne Bridge, being 30 ft. The preceding description of the Sagamore Bridge covers in a general way the structural features of both bridges.

Fenders.—All the bridges are protected with fender work. At the easterly end of the canal there are no destructive marine borers, and the fender for the Sagamore Bridge at this end of the canal was built of untreated wood. In the warm water of Buzzards Bay the marine borers are very destructive, and the fenders for the two bridges at this end of the canal are of creosoted pine. The approach portion of the fenders was designed and built in the ordinary manner with clusters of piles at the extreme ends and three rows of staggered piles in the approach proper. In the narrow throat immediately adjacent to the piers the common design was abandoned in order to obtain the maximum width between the fenders. This portion of the fender was built with 50-ft. span vertical trusses, in the case of both highway bridges, and with 60-ft. trusses for the railway bridge. These trusses were designed of sufficient strength to carry their own vertical load and with sufficient lateral stiffness to stand the rubbing impact of vessels. This lateral stiffness is provided by four courses of 6 by 12-in. timber wales, blocked, braced, and spliced, as shown by Fig. 13. The first and third courses from the channel side are extended and fastened to three of the dolphin piles to which collision impact is ultimately transmitted.

Fig. 13 shows the general design of the Buzzards Bay railway bridge fender, the design for the other bridges being similar.

Lighting.—The canal proper is lighted by 7-volt, 6.6-ampere, or 32-c-p., incandescent lights. These are carried by iron brackets on wooden poles, the poles being set at about the level of mean tide, with the plane of the lights 25 ft. above mean low water. The poles on the south side of the canal are painted red; those on the north side are painted black, thus following the rule for the colors of buoys entering a harbor, the canal being considered as an entrance to Boston Harbor. These lights are operated on an alternating current of about 500 volts, and are connected in series.

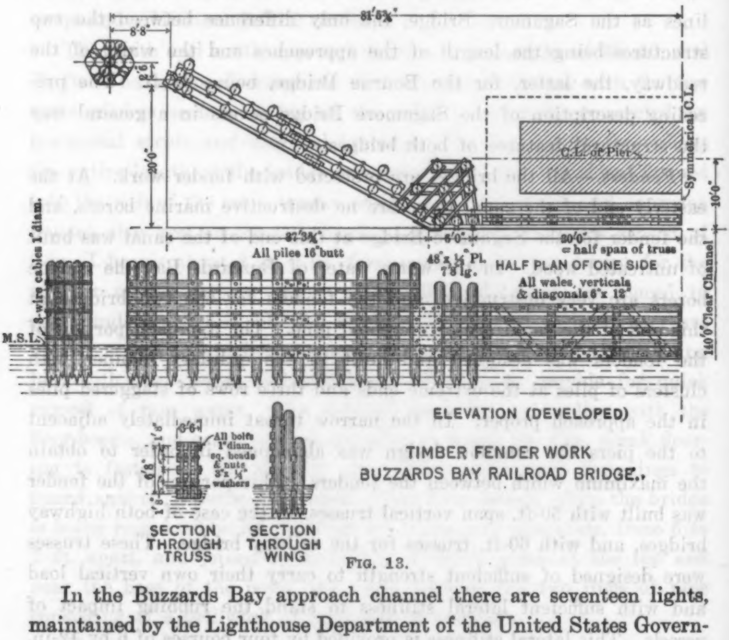


FIG. 13.

In the Buzzards Bay approach channel there are seventeen lights, maintained by the Lighthouse Department of the United States Government, the approach being in the open navigable waters of the United States. Four of these are on buoys at the western end of the channel and thirteen are on pile dolphins marking the channel from Wings Neck to Agawam Point. Coming from the west the lights on the port hand are white and on the starboard hand red. All these lights burn acetylene gas, flashing about 3 sec. duration and 2 sec. eclipse. On one of the two buoys at the western entrance there is a bell which

rings by wave action. In time of fog there are also three bells operated by storage batteries, the bells being maintained by the Canal Company.

On the north breakwater there is a 250-watt 111-volt incandescent light with a stereopticon globe, giving about 16 000 c-p. The plane of this light is 40 ft. above the water surface. At the eastern entrance there are two lights on buoys outside the breakwater, or about 400 ft. from its end, with a red flash, being luminous for 5 sec. and a gas and bell buoy 2½ miles northeast of the breakwater, showing a white flash every 6 sec., with duration of 2 sec., the light being 390 c-p.

Electrical Power.—The electrical power for the lighting of the canal and for the operation of the three bridges crossing it is purchased from the Southeastern Massachusetts Power and Electric Company, and is transmitted from steam plants at New Bedford or Plymouth. The power is transmitted at 22 000 volts, and can be supplied from the two sources by either of four routes to a step-down transformer sub-station near the west approach of the Bourne Highway Bridge. In the sub-station there are two 20-kw. alternating-current, gasoline-engine driven generating sets, which can be started on short notice for emergency conditions. From the sub-station 2 300-volt lines connect the three bridges, and, in addition, standard street lighting equipment is provided for the lights on each side of the canal.

At the western end of the Buzzards Bay Bridge there is a storage battery of 280 cells, of the chloride accumulator type, which is capable, at full charge, of supplying sufficient power to open and close the Buzzards Bay Bridge twenty-five or thirty times without replenishment.

In the operators' cages on each of the bridges there is a motor generator set for generating 550-volt direct current for operating the moving parts of the bridges and for charging the battery. In case of failure of the alternating current supplied over any one of the four transmission routes, power is available from the two gasoline-engine generating sets, or from the storage battery.

In addition to the foregoing sources of power, the street railway feeders of the New Bedford and Onset Street Railway Company can supply power at the Bourne and Buzzards Bay Bridges. Also, a special connection has been made to the Keith Car Company's plant at Sagamore, for the operation of this bridge in an emergency.

The present policy of operating in an emergency is to rely on the storage battery. There are separate connections from the battery house to the operator's cage on the Buzzards Bay Bridge, and, through special switching arrangements, the alternating-current lines connecting the three bridges are used in parallel for the positive side of the battery for the operation of the bridge motors, and the canal channel is used for the negative return. The capacity of the battery is ample to operate all three bridges many times without replenishment.

The power for operating the bridges and lighting the canal runs from 6 000 to 8 000 kw-hr. per month at a maximum peak load of about 140 kw.

The Buzzards Bay Bridge requires about 100 h.p. of maximum demand and about 1 min. to open or close. The power required for one operation is about 2 ampere-hours. The Bourne and Sagamore Bridges each require about 20 h.p. for the maximum demand and about the same time to open or close as the Buzzards Bay Bridge.

Engineering Personnel.—The responsible engineers of the company who have had charge of the work have been, of the Canal Company, W. J. Douglas, M. Am. Soc. C. E., Deputy Chief Engineer since August, 1915, and of the Construction Company, Eugene Klapp, M. Am. Soc. C. E., from the commencement of operations. The engineers in charge on the ground have been Henry W. Durham, M. Am. Soc. C. E., Resident Engineer from the commencement of operations to April 1st, 1912. Charles T. Waring, Assoc. M. Am. Soc. C. E., Resident Engineer from April 1st, 1912, to August 1st, 1914, when the canal was declared officially opened, although construction was not quite complete, and when Mr. Waring became Superintendent. From August 1st, 1914, to January 31st, 1915, the completion of construction was in charge of A. S. Ackerman, Assoc. M. Am. Soc. C. E., previously Assistant Engineer, when the duties were re-assumed by Mr. Waring in connection with his other duties as Superintendent, and which he retained until January 30th, 1916. Mr. W. S. Crocker, who was Assistant Engineer during construction, became Resident Engineer under the operating management.

To these gentlemen and to their assistants the Chief Engineer expresses his obligations, and, in the preparation of this paper, particularly to Mr. Eugene E. Halmos, member of the staff, for his co-

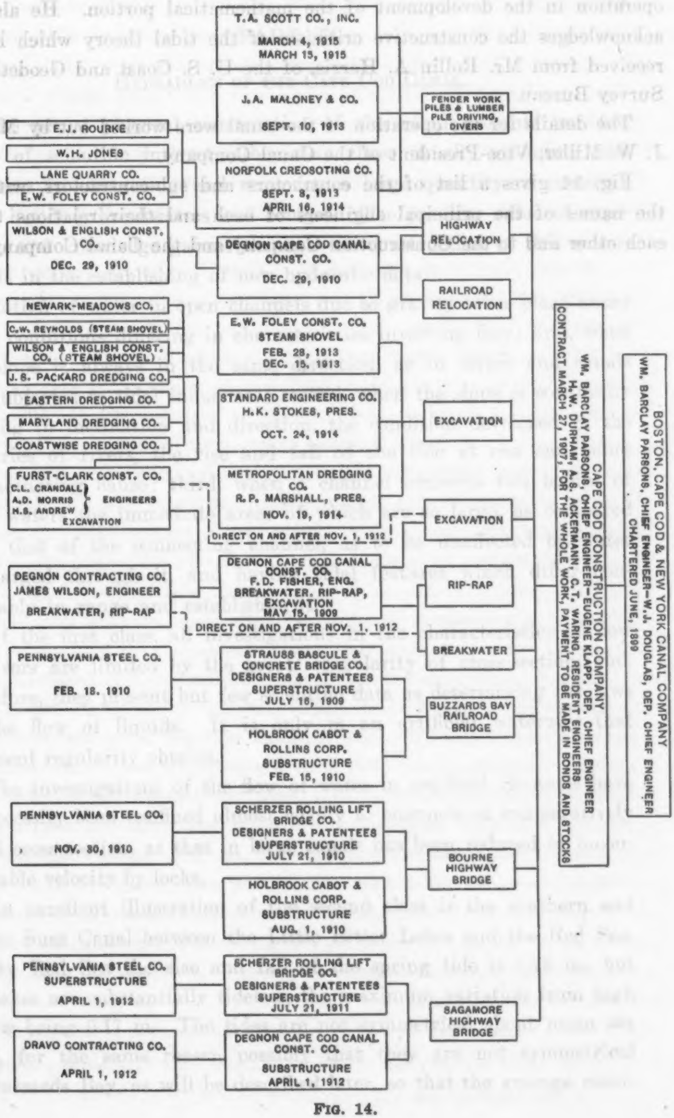

T.A. SCOTT CO., INC.
MARCH 4, 1915
MARCH 15, 1915
J.A. MALONEY & CO.
SEPT. 10, 1913
FENDER WORK
PILES & LUMBER
PILE DRIVING,
DIVERS
E.J. ROURKE
W.H. JONES
LANE QUARRY CO.
E.W. FOLEY CONST. CO.
NORFOLK CREOSOTING CO.
SEPT. 8, 1913
APRIL 16, 1914
WILSON & ENGLISH CONST. CO.
DEC. 29, 1910
HIGHWAY RELOCATION
DEGNON CAPE COD CANAL CONST. CO.
DEC. 29, 1910
RAILROAD RELOCATION
NEWARK-MEADOWS CO.
C.W. REYNOLDS (STEAM SHOVEL)
WILSON & ENGLISH CONST. CO. (STEAM SHOVEL)
J.S. PACKARD DREDGING CO.
EASTERN DREDGING CO.
MARYLAND DREDGING CO.
COASTWISE DREDGING CO.
E.W. FOLEY CONST. CO.
STEAM SHOVEL
FEB. 28, 1913
DEC. 15, 1913
STANDARD ENGINEERING CO.
H.K. STOKES, PRES.
OCT. 24, 1914
EXCAVATION
FURST-CLARK CONST. CO.
C.L. CRANDALL
A.D. MORRIS
H.S. ANDREW
ENGINEERS
EXCAVATION
METROPOLITAN DREDGING CO.
R.P. MARSHALL, PRES.
NOV. 19, 1914
EXCAVATION
DIRECT ON AND AFTER NOV. 1, 1912
DEGNON CONTRACTING CO.
JAMES WILSON, ENGINEER
BREAKWATER, RIP-RAP
DEGNON CAPE COD CANAL CONST. CO.
F.D. FISHER, ENG.
BREAKWATER, RIP-RAP, EXCAVATION
MAY 15, 1909
RIP-RAP
DIRECT ON AND AFTER NOV. 1, 1912
BREAKWATER
PENNSYLVANIA STEEL CO.
FEB. 18, 1910
STRAUSS BASCULE & CONCRETE BRIDGE CO.
DESIGNERS & PATENTEES
SUPERSTRUCTURE
JULY 16, 1909
BUZZARDS BAY RAILROAD BRIDGE
HOLBROOK CABOT & ROLLINS CORP.
SUBSTRUCTURE
FEB. 15, 1910
PENNSYLVANIA STEEL CO.
NOV. 30, 1910
SCHERZER ROLLING LIFT BRIDGE CO.
DESIGNERS & PATENTEES
SUPERSTRUCTURE
JULY 21, 1910
BOURNE HIGHWAY BRIDGE
HOLBROOK CABOT & ROLLINS CORP.
SUBSTRUCTURE
AUG. 1, 1910
PENNSYLVANIA STEEL CO.
SUPERSTRUCTURE
APRIL 1, 1912
SCHERZER ROLLING LIFT BRIDGE CO.
DESIGNERS & PATENTEES
SUPERSTRUCTURE
JULY 21, 1911
SAGAMORE HIGHWAY BRIDGE
DRAVO CONTRACTING CO.
APRIL 1, 1912
DEGNON CAPE COD CANAL CONST. CO.
SUBSTRUCTURE
APRIL 1, 1912
CAPE COD CONSTRUCTION COMPANY
WM. BARCLAY PARSONS, CHIEF ENGINEER—EUGENE KLAPP, DEP. CHIEF ENGINEER
H.W. DURHAM, A.S. ACKERMANN, C.T. WARING, RESIDENT ENGINEERS
CONTRACT MARCH 27, 1907, FOR THE WHOLE WORK, PAYMENT TO BE MADE IN BONDS AND STOCKS
BOSTON, CAPE COD & NEW YORK CANAL COMPANY
WM. BARCLAY PARSONS, CHIEF ENGINEER—W.J. DOUGLAS, DEP. CHIEF ENGINEER
CHARTERED JUNE, 1899

FIG. 14.

operation in the development of the mathematical portion. He also acknowledges the constructive criticism of the tidal theory which he received from Mr. Rollin A. Harris, of the U. S. Coast and Geodetic Survey Bureau.

The details for the operation of the canal were worked out by Mr. J. W. Miller, Vice-President of the Canal Company.

Fig. 14 gives a list of the contractors and sub-contractors, with the names of the principal engineers of each and their relations to each other and to the Construction Company and the Canal Company.

PART II.

Hydraulics of the Cape Cod Canal.

The actual physical construction of the canal presents but few items of scientific interest, other than those usually attending new problems in excavating and removing a large quantity of material, or in building breakwaters and bridges. The peculiar value of experience and knowledge gained through the construction of this waterway is in the establishing of new hydraulic data.

Motion of water in open channels due to gravity takes place under three conditions differing in characteristics involving flow: first, when the slope is always in the same direction, as in rivers and canals not subjected to tidal influences; second, when the slope is constantly varying in inclination and direction, the condition developed in the estuaries of rivers, the rise and fall of the tide at one end being the actuating cause; third, when a channel connects two bodies of tidal waters the immediate areas of which are so large, as compared with that of the connecting channel, as to be unaffected by water discharged through it, and having tidal features which differ considerably in range and establishment.

Of the first class, all investigations in the characteristics of flow in rivers are limited by the great irregularity of cross-section, and, therefore, they present but few scientific data as determining the laws of the flow of liquids. It is only in an artificial waterway that sufficient regularity obtains.

The investigations of the flow of water in artificial channels have of necessity been confined almost wholly to channels of comparatively small cross-section, as that in large canals has been reduced to inconsiderable velocity by locks.

An excellent illustration of the second class is the southern end of the Suez Canal between the Little Bitter Lakes and the Red Sea. In the Red Sea the rise and fall of the spring tide is 1.75 m., but the lakes are substantially tideless, the maximum variation from high to low being 0.17 m. The tides are not symmetrical about mean sea level, for the same reason possibly that they are not symmetrical in Buzzards Bay, as will be described later, so that the average maxi-

mum slopes are 0.032 and 0.027 m. per km., northerly and southerly, respectively, the length of uniform canal section being 37.05 km.*

Another illustration is the Pamlico Sound-Beaufort Canal, in North Carolina, where Earl I. Brown, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. A., made some interesting current observations.† This canal has a comparatively small cross-section (90 by 10 ft.), with an average tidal oscillation of 3.8 ft. at Beaufort Harbor. The non-tidal end, however, is greatly affected by the action of the wind in Pamlico Sound.

Of the third class, the Cape Cod Canal is the only large instance of such a channel where all the conditions are conducive to scientific study.

Of the natural waterways of this class, the East River, connecting Long Island Sound and New York Bay, is perhaps the best example. Extensive and careful observations‡ in 1912 by W. M. Black, M. Am. Soc. C. E., Colonel, Corps of Engineers, U. S. A. (now Brig-Gen. and Chief of Engineers), show similar characteristics between the motion of water in that stream and in the Cape Cod Canal, but the irregularity of cross-section of the river renders the making of analytical study of observed results impossible. The Kaiser Wilhelm Canal, popularly known as the Kiel Canal, also connects large bodies of tidal waters, but it is equipped with locks, so that flow through the canal, other than local drainage, is prevented.

Cape Cod Bay and Buzzards Bay, which the canal connects, are large sheets of open water. Cape Cod Bay is really a directly connecting part of the Atlantic Ocean, the opening being about 20 miles across. The bay is circular, with a uniformly curved and substantially unbroken shore line, and with considerable depth of water, ranging from 12 to 25 fathoms, over a flat floor with no abrupt hollows or ridges. Buzzards Bay, on the other hand, is much longer than it is wide, being 50 miles long and having an average width of about 7 miles, with an average depth of about 10 fathoms at its opening, shoaling to about 2 fathoms at the upper end, except in the original narrow channels to Monument Beach and Wareham, or local shoals, where the depth is still less.

* The facts regarding the tides and currents are contained in the reports of the "Commission Consultative Internationale des Travaux" for 1906 and 1907.

† The results are published in the March–April, 1912, number of *Professional Memoirs.*

‡ *Professional Memoirs,* June, 1913.

These bays are affected by two different tidal waves, which are quite dissimilar in their establishments, ranges, and other characteristics. The wave that passes up Buzzards Bay is a branch of the great Atlantic wave which runs northward along the coast, its velocity of travel being somewhat retarded in its passage over the comparatively shoal water along the coast. The wave in Cape Cod Bay is another branch of the same great wave, which, traveling faster in the deeper water off shore, strikes the Nova Scotia coast and thence is deflected in part westward and then southward. These two waves meet and interfere in Vineyard Sound, and, in the irregularity produced by such interference, add to the difficulties of navigation and cause fogs owing to their difference in temperature. The mean range of the tide in Cape Cod Bay is 9.0 ft. at Boston, 9.6 ft. at Plymouth, 8.9 ft. at the entrance of the canal,* and 9.2 ft. at Provincetown, whereas the mean range in Buzzards Bay is 4.0 ft. at New Bedford and 3.6* ft. at the entrance to the canal.

Briefly, then, the problem developed by the construction of the Cape Cod Canal is an extremely complex one in hydrodynamics, being the analysis of the motion of water in a canal of considerable magnitude connecting two seas, the tides in which differ to a great extent, both as to phase and amplitudes.

Before entering on the analysis of the problem, the basal facts, as determined by the records, should be set forth.

In the early days, when a canal was under consideration, the known variation in tidal head at the two ends of the canal precluded any consideration of a sea-level canal through an unanalyzed fear of disastrous results, and it was not until 1861 that any study was made of the tidal phenomena and conditions, when a legislative committee undertook the investigation. As explained previously, the committee called to its aid Gen. Totten, Professor Bache, and Commander Davis, and these gentlemen in turn placed Mr. Henry Mitchell in charge of the tidal observations. Mr. Mitchell's reports are set forth in detail in the general report of the committee, published as a State document in 1864. He carried these observations over one month, therefore covering a lunar cycle, and found that the mean rise and fall was 9.17 and 4.11 ft. at the east and west ends, respectively, results that agree closely with those determined by observations

* From records obtained subsequent to the opening of the canal.

extending over some years, namely, 9.02* and 3.91* ft.; but his accuracy ended there, because, knowing that the two tidal waves oscillated uniformly above and below mean sea level at points along the coast, according to his own and other determinations for the coast survey, he assumed that the same conditions obtained in the locality under investigation. Apparently, he set up two tide recording stations, where he ascertained the heights of low and high waters and computed mean sea level in each case as half way between mean high and mean low. He omitted to run a line of check levels across the isthmus to determine whether his computed mean sea levels were actually, as he assumed them to be, the same datum plane—another illustration of the danger of making assumptions in engineering work without actual knowledge. Therefore, he determined that though the durations of the flood and ebb tides were the same in Cape Cod Bay, in Buzzards Bay the flood endured for 7 hours 01 min. and the ebb for 5 hours 23 min., and, as a corollary, that the maximum differences in simultaneous tidal heights between the two ends on the flood and on the ebb were not nearly equal. As indicative of the lack of knowledge then existing regarding the flow of water in large open channels, the tidal report contained a conclusion, which the board thought of sufficient importance to put in Italics, reading: "The greater the transverse section of the free canal, the more rapid will be the flow through it." It seems extraordinary that as recently as 1860 that statement was sufficiently novel to be deemed, by men like Professor Bache and Mr. Henry Mitchell, as "interesting" and worthy of being put in Italics in a Government report.

Tidal observations were begun by the writer in October, 1907, when automatic recording mareograph instruments, of the U. S. Coast Survey type, were set up at Monument Beach in Buzzards Bay and at Barnstable in Cape Cod Bay. When the canal excavation began, these instruments were transferred to the points just within the canal at each end (Canal Stations 35 and 380), where they have been maintained continuously under observation. The differences in tidal heights and times between the first selected points and those within the canal were found to be so small as to be negligible. Accurate levels were run between the two instruments, so that their

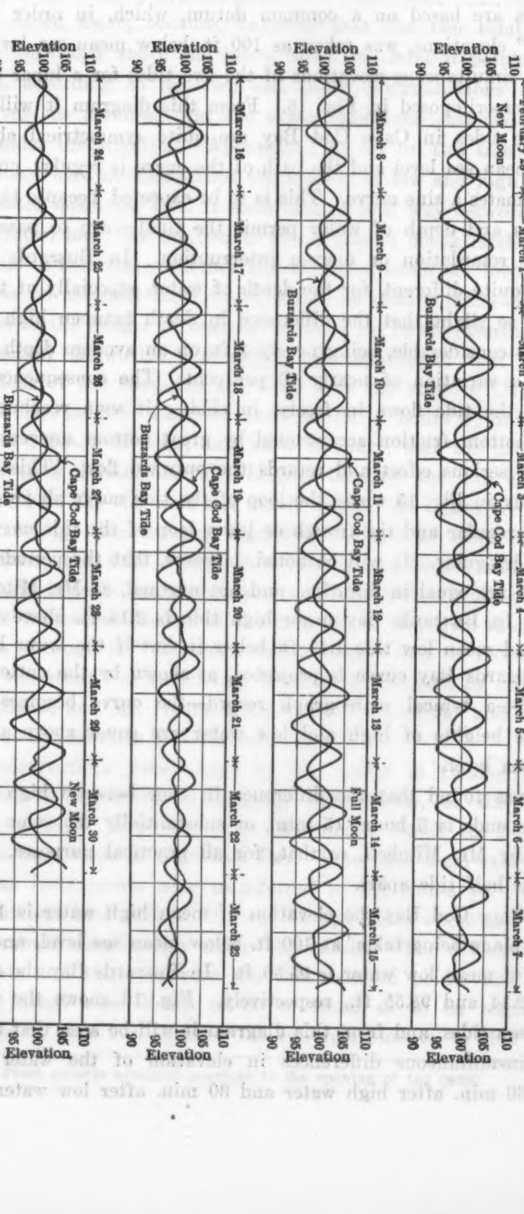
* From records obtained previous to the opening of the canal.

readings are based on a common datum, which, in order to avoid "minus" elevations, was taken as 100 ft. below mean sea level.

The simultaneous elevations of the two tides for a lunar cycle are shown superimposed in Fig. 15. From this diagram it will be seen that the tides in Cape Cod Bay are quite symmetrical above and below mean sea level and the path of the curve is regular, and closely approximates a sine curve. This is to be expected, because the smooth contours and depth of water permit the tidal wave to pass without sensible retardation or uneven interruption. In Buzzards Bay the case is quite different, for the depth of water, especially at the upper end, is so slight that the difference in depth between high and low water is considerable, being nearly 4 ft. on an average depth of about 10 ft., a variation of nearly 40 per cent. The consequence is that though the tide flows in freely, in ebbing it soon reaches a point where bottom friction accentuated by great bottom unevenness, produces a serious effect and retards the outward flow. This is shown distinctly in Fig. 15 where the loop of the tide curve above mean sea level is regular and the trough or lower loop of the tide curve is distinctly irregular. It will be noted, however, that the periods of flood and ebb are equal in duration, and not unequal, as Mr. Mitchell supposed. In Buzzards Bay mean high tide is 2.14 ft. above mean sea level and mean low tide 1.45 ft. below it, but if the lower branch of the Buzzards Bay curve is projected, as shown by the dotted line in Fig. 16—a typical mareograph record—the curve becomes regular, and the heights of high and low water are equal above and below mean sea level.

It was found that the difference in time between high water at the two ends is 3 hours 15 min., or substantially the same as determined by Mr. Mitchell, so that, for all practical purposes, the tides are just half tide apart.

In Cape Cod Bay the elevation of mean high water is 104.42 ft., datum plane being taken at 100 ft. below mean sea level, and the elevation of mean low water is 95.50 ft. In Buzzards Bay the elevations are 102.14 and 98.55 ft., respectively. Fig. 16 shows the curves of two mean tides, and from this diagram it will be seen that the maximum instantaneous differences in elevation of the water surfaces occur 30 min. after high water and 30 min. after low water in Cape



MAREOGRAPH FOR A WHOLE LUNAR CYCLE, FEBRUARY 28TH TO MARCH 30TH, 1911

Fig. 15.

Cod Bay, and that for 1 hour 30 min. in each case the difference in elevation is practically unchanged, the paths of the two curves being nearly parallel. It is fortunate, from the scientific standpoint, that the incompleteness or irregularity of the trough of the Buzzards Bay tide curve does not affect the maximum differences, as the irregularity does not begin until after the instantaneous difference has begun to diminish sensibly. The irregularity, therefore, affects only hydrodynamics and flow conditions that are less than the maximum.

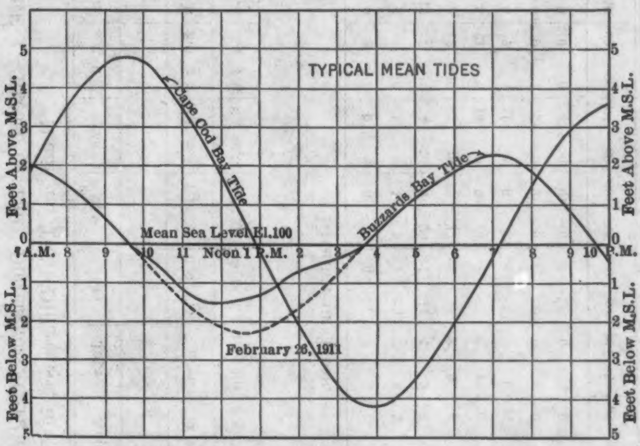


FIG. 16.

As the slope remains practically constant for a time long enough to eliminate variations produced by wave propagation, the canal becomes an example of motion of water affected by variations in elevations at both ends, and also of a large channel in which the "head" is constant.

In actuality, the tidal elevations vary considerably from the mean figures, according to the moon phases producing "spring" and "neap" tides and also to irregularities following wind action. Tables 2 to 9, inclusive, give the numbers of tides in each year grouped in variations of foot differences, from 1908 to 1915, inclusive. The average maximum differences, as determined from the data contained in these tables, are 5 ft. producing easterly and 5.3 ft. producing westerly currents, including the effects of storms. The maximum difference occurred on January 13th, 1915, when it was 9.5 ft. westerly, being

TABLE 2.—Record of Tidal Differences, Cape Cod Canal, for 1908.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1908.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum......	8.6	8.5	9.2	7.6	8.0	8.8	7.0	8.5	7.1	7.2	7.0	7.5	7.5	8.1	7.0	8.0	6.8	8.0	8.4	8.2	7.6	7.3	7.3	7.4	9.2	8.8
Minimum......	1.3	3.5	3.3	4.4	3.2	2.8	2.9	3.3	3.2	3.7	3.4	3.4	3.8	4.0	3.6	3.6	2.5	3.3	2.1	3.8	2.6	2.9	3.2	3.5	1.3	2.8

East Current +
West " —

Max. Diff.	Number of Tides.																									
8 +	2	1	4	0	2	6	0	2	0	0	0	0	0	2	0	1	0	2	4	1	0	0	0	0	12	15
7 +	0	9	9	4	8	8	1	6	1	3	1	5	5	4	2	7	0	7	6	11	1	8	1	4	33	76
6 +	12	15	6	19	7	19	4	16	9	20	11	11	7	13	6	8	11	15	8	10	10	13	10	17	101	176
5 +	14	21	20	28	13	16	21	19	25	17	22	26	26	22	6	23	12	13	17	23	18	17	16	24	210	249
4 +	23	11	13	5	21	6	19	10	19	13	18	9	21	19	33	19	16	17	18	14	10	15	21	12	232	150
3 +	7	3	4	0	9	4	11	5	6	6	6	7	3	0	13	2	17	4	5	1	16	2	11	3	108	37
2 +	0	0	0	0	0	1	2	0	0	0	0	0	0	0	0	0	2	0	2	0	3	3	0	0	9	4
1 +	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0
	60	60	56	56	60	60	58	58	60	59	58	58	60	60	60	60	58	58	60	60	58	58	59	60	707	707

TABLE 3.—RECORD OF TIDAL DIFFERENCES, CAPE COD CANAL, FOR 1909.

Maximum Tidal Differences, in Feet.

	JAN.		FEB.		MAR.		APR.		MAY.		JUNE.		JULY.		AUG.		SEPT.		OCT.		NOV.		DEC.		1909.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum.....	6.8	8.0	6.8	7.8	5.8	7.7	6.0	6.3	5.6	6.7	5.6	5.4	5.5	5.4	7.8	7.4	8.2	8.2	8.1	8.3	7.8	9.2	6.8	8.3	8.2	9.2
Minimum......	2.2	3.3	2.5	2.5	1.7	3.3	2.0	2.8	2.9	2.3	3.4	2.6	3.2	2.3	2.6	3.4	3.8	3.0	2.7	3.3	2.9	3.8	3.2	4.2	1.7	2.3
East Current + West " —																										
MAX. DIFF.	NUMBER OF TIDES.																									
8 +	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	1	1	2	0	6	0	2	4	12
7 +	0	2	0	5	0	6	0	0	0	0	0	0	0	0	2	1	6	8	6	9	5	5	0	8	19	44
6 +	1	12	1	11	0	5	1	3	0	3	0	0	0	0	4	4	11	7	10	8	9	7	9	21	46	81
5 +	12	23	14	16	10	22	11	14	5	9	7	4	14	5	14	18	20	24	24	23	18	17	23	19	172	194
4 +	22	16	16	19	19	20	25	24	31	27	30	36	28	28	35	25	17	14	13	15	17	21	25	10	278	255
3 +	18	6	19	2	28	7	18	14	23	18	21	17	18	23	4	12	1	3	5	3	8	2	3	0	166	107
2 +	7	0	4	1	2	0	3	3	1	3	0	1	0	4	1	0	0	0	1	0	1	0	0	0	20	12
1 +	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	57	60	60	58	58	60	60	706	705

TABLE 4.—Record of Tidal Differences, Cape Cod Canal, for 1910.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1910.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum......	7.2	7.1	7.8	7.2	7.5	7.1	8.0	7.6	8.5	7.6	8.2	7.3	7.9	7.0	7.8	7.0	8.2	7.1	8.8	8.2	8.7	8.5	8.2	8.5	8.8	8.5
Minimum......	1.3	3.1	2.2	1.7	3.3	2.3	3.4	2.4	3.7	3.8	4.2	4.0	4.1	3.6	3.8	3.2	3.4	2.9	2.8	3.1	3.1	3.4	3.6	3.0	1.3	1.7
East Current + West " —																										
Max. Diff.	**Number of Tides.**																									
8 +	0	0	0	0	0	0	1	0	4	0	1	0	0	0	0	0	3	0	6	1	3	3	3	1	21	5
7 +	1	3	3	4	2	4	4	8	8	8	4	3	10	1	10	2	9	7	6	9	7	8	5	4	69	61
6 +	17	10	12	14	8	8	12	6	6	7	16	13	25	19	19	18	19	16	15	13	11	14	9	16	169	154
5 +	17	27	16	15	22	14	15	15	24	21	22	14	15	25	19	25	14	18	18	18	19	17	20	24	221	233
4 +	13	10	16	13	2	18	17	13	16	20	15	28	10	12	11	11	8	11	7	13	16	11	21	8	170	168
3 +	9	10	5	5	8	13	9	14	2	4	0	0	0	3	1	4	5	5	7	6	2	5	2	7	50	76
2 +	2	0	2	2	0	3	0	2	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	5	8
1 +	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	58	60	60	58	58	60	60	706	706

TABLE 5.—RECORD OF TIDAL DIFFERENCES, CAPE COD CANAL, FOR 1911.

Maximum Tidal Differences, in Feet.

	JAN.		FEB.		MAR.		APR.		MAY.		JUNE.		JULY.		AUG.		SEPT.		OCT.		NOV.		DEC.		1911.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum.....	7.7	7.7	8.9	7.5	6.9	7.2	7.1	7.7	7.2	7.4	6.6	7.4	7.3	7.3	6.8	7.5	6.8	7.1	7.0	7.2	7.8	7.2	7.8	7.9	8.9	7.9
Minimum......	3.5	2.8	2.1	2.3	2.3	3.2	2.2	3.3	3.2	3.6	3.3	4.3	3.7	4.2	3.4	3.5	1.6	3.1	1.8	2.8	2.4	3.4	2.7	4.0	1.6	2.3

East Current +
West " —

MAX. DIFF.	NUMBER OF TIDES.																									
8 +	0	0	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	0
7 +	5	5	8	1	0	3	1	4	2	5	0	4	2	2	0	3	0	2	2	3	2	2	4	3	26	37
6 +	18	17	8	6	18	24	10	14	10	12	5	12	9	11	11	11	12	19	6	18	7	14	4	18	108	171
5 +	21	23	7	18	21	15	17	20	11	19	16	29	16	32	25	27	16	17	17	18	20	18	18	26	205	262
4 +	15	10	9	20	12	14	18	13	23	21	26	13	31	15	18	15	17	11	17	11	16	19	24	18	226	180
3 +	6	4	15	6	10	4	9	7	14	3	11	0	2	0	6	4	12	9	13	8	11	5	9	0	118	50
2 +	0	1	1	3	4	0	3	0	0	0	0	0	0	0	0	0	0	0	4	2	2	0	1	0	15	6
1 +	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	2	0
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	58	60	60	58	58	60	60	706	706

TABLE 6.—Record of Tidal Differences, Cape Cod Canal, for 1912.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1912.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum......	7.4	8.7	7.2	8.0	8.0	8.2	6.4	7.5	6.9	8.1	7.7	7.3	8.1	7.6	7.9	7.9	8.9	8.5	8.5	7.5	7.8	7.6	7.6	7.4	8.9	8.7
Minimum......	1.9	3.7	2.3	3.5	2.5	3.3	2.6	3.1	2.1	3.1	2.9	3.9	3.6	1.9	3.7	3.8	2.2	2.9	2.9	3.1	2.9	2.6	2.9	2.5	1.9	1.9

East Current +
West " —

Max. Diff.	Number of Tides.																									
8 +	0	3	0	2	1	2	0	0	0	1	0	0	1	0	0	0	2	2	2	0	0	0	0	0	6	10
7 +	4	2	2	2	7	4	0	3	0	1	2	4	2	4	7	8	5	7	2	4	6	3	3	2	40	44
6 +	7	11	8	14	8	25	3	14	10	14	6	16	9	11	7	9	9	14	9	15	11	11	8	7	95	161
5 +	9	30	15	20	19	12	22	22	25	25	24	19	16	21	17	28	8	20	17	17	19	17	25	23	216	254
4 +	14	13	18	15	14	11	20	15	15	13	22	17	28	21	26	12	17	9	22	14	16	20	16	18	228	178
3 +	21	1	12	3	8	6	10	4	7	6	3	1	4	2	3	3	14	5	7	10	5	6	6	9	100	56
2 +	4	0	1	0	3	0	3	0	3	0	1	0	0	1	0	0	3	1	1	0	1	1	1	1	21	4
1 +	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
	60	60	56	56	60	60	58	58	60	60	58	57	60	60	60	60	58	58	60	60	58	58	59	60	707	707

TABLE 7.—RECORD OF TIDAL DIFFERENCES, CAPE COD CANAL, FOR 1913.
Maximum Tidal Differences, in Feet.

	JAN.		FEB.		MAR.		APR.		MAY.		JUNE.		JULY.		AUG.		SEPT.		OCT.		NOV.		DEC.		1913.	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum.....	8.2	7.8	8.2	8.3	7.9	8.0	6.8	7.6	6.6	7.1	6.2	7.2	7.1	6.8	7.0	7.5	7.7	7.9	7.2	8.0	6.9	7.5	7.1	7.3	8.2	8.3
Minimum......	3.1	2.6	3.4	3.5	2.6	2.5	2.4	3.4	1.9	3.6	3.4	3.6	3.8	3.6	3.6	3.5	3.2	3.5	2.9	3.4	2.7	3.6	2.0	3.6	1.9	2.5

East Current +
West " —

MAX. DIFF.	NUMBER OF TIDES.																									
8 +	1	0	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	2	3
7 +	6	6	2	9	7	3	0	5	0	3	0	2	1	0	1	5	7	7	3	13	0	2	1	1	28	56
6 +	8	10	8	7	6	10	8	12	4	12	4	10	8	13	12	10	7	11	9	14	6	13	3	17	83	139
5 +	18	17	14	10	12	20	15	18	12	24	25	27	25	23	22	21	18	18	10	10	17	22	26	26	214	286
4 +	23	18	20	22	21	13	19	17	28	18	24	16	21	22	23	20	18	15	26	15	22	17	21	12	266	205
3 +	4	7	9	5	12	11	14	6	13	3	5	3	5	2	2	4	8	7	11	6	10	4	7	4	100	62
2 +	0	2	0	0	2	2	2	0	2	0	0	0	0	0	0	0	0	0	1	0	3	0	2	0	12	4
1 +	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	58	60	59	58	58	60	60	706	705

TABLE 8.—Record of Tidal Differences, Cape Cod Canal, for 1914.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1914.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum.....	7.0	7.7	8.1	8.3	7.1	8.4	7.1	7.6	7.7	8.3	7.5	7.7	6.5	7.0	6.8	6.8	7.8	7.0	7.0	7.0	7.5	7.1	7.0	7.6	8.1	8.4
Minimum......	3.3	2.5	2.7	2.9	2.3	3.6	2.2	2.8	3.2	3.2	3.8	3.8	3.2	3.4	2.9	2.9	2.1	3.0	2.5	3.1	2.8	2.8	2.5	2.6	2.1	2.5

East Current +
West " —

Number of Tides.

Max. Diff.																										
8 +	0	0	1	1	0	4	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7
7 +	2	5	2	1	1	9	3	2	2	6	2	3	0	1	0	0	1	1	3	3	3	2	2	2	21	33
6 +	12	11	6	13	9	5	4	8	10	9	7	14	3	9	3	9	6	14	6	9	2	4	7	10	75	115
5 +	24	23	14	19	17	22	14	19	13	21	22	22	21	34	18	25	17	22	9	17	13	27	21	17	208	268
4 +	18	11	13	13	20	17	19	21	22	18	24	16	23	12	20	17	17	13	22	22	23	20	16	20	237	200
3 +	4	9	14	6	8	3	15	7	13	4	3	3	12	4	18	8	13	8	17	9	15	4	11	10	143	115
2 +	0	3	4	1	5	0	3	1	0	0	0	0	0	0	1	1	4	0	3	0	2	1	3	1	25	8
1 +	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	60	60	54	54	60	60	58	58	60	60	58	58	59	60	60	60	58	58	60	60	58	58	60	60	705	706

TABLE 9.—RECORD OF TIDAL DIFFERENCES, CAPE COD CANAL, FOR 1915.

Maximum Tidal Differences, in Feet.

	JAN.		FEB.		MAR.		APR.		MAY.		JUNE.		JULY.		AUG.		SEPT.		OCT.		NOV.		DEC.		1915.	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum.....	8.0	9.5*	6.1	7.5	6.5	7.1	7.6	7.1	7.8	6.6	8.0	6.8	7.0	6.5	6.7	6.6	7.0	6.2	6.0	6.2	8.3	6.6	7.5	7.2	8.3	9.5*
Minimum......	2.9	2.4	2.5	2.5	2.6	2.2	2.0	2.0	3.2	2.9	3.9	3.4	3.7	2.9	2.6	2.7	2.7	2.6	2.1	2.4	2.8	1.7	3.0	3.0	2.0	1.7
East Current + West " —																										
MAX. DIFF.	NUMBER OF TIDES.																									
8 +	1	1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	2	0	0	0	4	1
7 +	5	0	0	1	0	2	5	2	2	0	2	0	1	0	0	0	5	0	0	0	6	0	6	1	32	6
6 +	7	2	3	13	6	7	4	6	14	5	6	2	5	6	6	1	7	4	1	2	5	9	5	2	69	59
5 +	18	15	17	20	17	18	24	21	24	13	20	9	22	19	25	23	19	20	18	15	20	9	11	10	235	192
4 +	21	31	16	12	17	17	22	14	16	30	28	30	27	27	20	20	13	18	23	19	12	23	24	32	239	273
3 +	7	8	14	5	13	12	2	9	4	10	1	17	5	7	7	14	13	13	13	18	12	16	13	15	104	144
2 +	1	2	4	4	7	3	1	6	0	2	0	0	0	1	2	2	1	3	5	6	1	0	0	0	22	29
1 +	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1
	60	59	54	55	60	59	58	58	60	60	58	58	60	60	60	60	58	58	60	60	58	58	59	60	705	705

* Storm—Jan. 13th, 1915.

produced by a great tide on the Massachusetts coast, the result of a violent storm. The minimum difference was 1 ft. 3 in. easterly. It will be seen, on study of these tables, that the excessive tides are comparatively few, the greater majority, about 83.5% in fact, giving heads that vary from 3 to 6 ft., and only about 2% giving differences

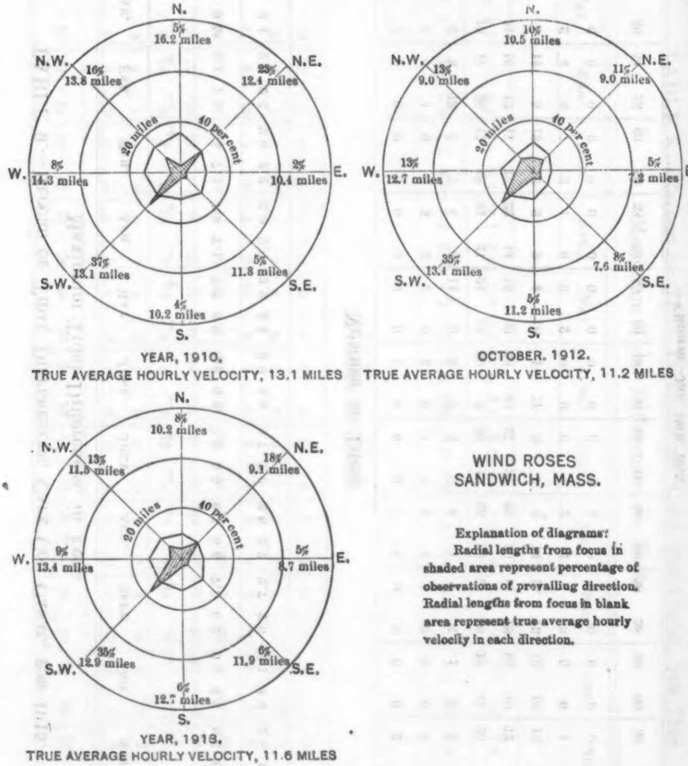


FIG. 17.

exceeding 7 ft. In these extreme cases the maximum difference lasts for a few minutes only, as the tidal curves no longer maintain their parallelism.

The difference in elevation at any instant is the hydraulic head that produces the velocity of the current, and as it occurs in a length of canal of 34 500 ft. (the distance apart of the automatic tide gauges),

the resulting mean maximum slope is 0.000154. It is interesting to point out that this slope is nearly four times as steep as the maximum slope that would have obtained at the Panama Canal had it been built at sea level, taking the maximum tidal oscillation at Panama at 22 ft., only one-half of which would have been above or below mean sea level, and neglecting the negligibly small tidal difference at Colon.

In order to ascertain the action of the wind, a recording anemometer was established at the same time as the tide gauges. Monthly wind roses for selected typical years are shown in Fig. 17, which are self-explanatory. Table 10 gives a record of winds having a velocity which exceeded 40 miles per hour between 1908 and 1915, inclusive. The prevailing winds are northeast and southwest; they blow across the canal, and produce no direct effect.

TABLE 10.—TRUE WIND VELOCITIES OF MORE THAN 40 MILES PER HOUR, FROM 1908 TO DECEMBER, 1915.

Date.	Time.	Total movement, in miles.	Duration, in hours.	Maximum velocity, in miles per hour.	Mean velocity, in miles per hour.	Direction.
Jan. 24, 1908...	11.00 A. M.—12.00 M.	40	1	40.8	40.0	N. E.
Dec. 26, 1909...	3.30 A. M.— 9.30 P. M.	799	18	55.2	44.4	N. W.
Jan. 14, 1910...	6.15 P. M.—10.15 P. M.	160	4	42.2	40.0	N. E.
Jan. 15, 1910...	6.00 A. M.— 4.30 P. M.	436	10.5	45.1	41.5	N.
Feb. 1, 1910...	8.45 A. M.— 1.00 P. M.	193	4.25	41.5	40.8	N. W.
Feb. 4, 1910...	10.00 A. M.— 3.00 P. M.	215	5	45.1	43.0	N.
Feb. 18, 1910...	1.30 A. M.— 2.45 P. M.	52	1.25	43.7	41.5	N. W.
Nov. 27, 1910...	7.15 A. M.—10.15 A. M.	122	3	40.8	40.8	N. W.
Mar. 16, 1911...	7.00 A. M.— 9.30 A. M.	100	2.5	43.0	40.0	W
July 29, 1911...	3.30 P. M.— 5.30 P. M.	84	2	45.1	42.2	S. W.
Dec. 28, 1911...	1.15 P. M.— 2.15 P. M.	40	1	45.1	40.0	W.
Feb. 22, 1912...	9.00 A. M.— 1.30 P. M.	190	4.5	48.0	42.2	S. W.
Mar. 15, 1912...	8.30 P. M.— 9.30 P. M.	40	1	45.1	40.0	W.
Jan. 3, 1913...	10.30 P. M.—10.45 P. M.	13	0.25	42.2	42.2	S. E.
Jan. 4, 1913...	10.45 P. M.— 3.00 A. M.	222	4.25	45.1	40.0	S. W.
Mar. 24, 1913...	1.10 P. M.— 3.00 P. M.	93	1.83	42.2	41.3	S. W.
Mar. 27, 1913...	7.45 A. M.— 4.40 P. M.	443	8.9	46.6	40.5	S.
Jan. 12, 1914...	4.15 P. M.— 5.25 P. M.	59	1.16	45.1	41.5	S. W.
Jan. 13, 1914...	1.00 A. M.— 1.15 A. M	13	0.25	41.5	41.5	N. W.
Mar. 1, 1914...	3.45 P. M.— 4.30 P. M.	36	0.75	48.0	47.3	S. E.
Dec. 13, 1914...	11.35 P. M.—12.00 M.	23	0.58	40.8	40.0	S. E.
Dec. 14, 1914...	12.00 M. — 2.05 A. M.	85	2.08	42.2	40.8	N. E.
Nov. 5, 1915...	8.05 P. M.— 8.30 P. M.	17	0.41	40.8	40.8	N. W.
Dec. 13, 1915...	5.00 P. M.— 7.30 P. M.	113	2.50	50.0	45.0	N. E.

In order to determine whether any appreciable difference in tidal conditions was produced by the flow of water through the canal, the tidal records were taken for four calendar months prior to the opening of the canal and during the same months after the opening. An average of all the tides in this period is given in Table 11.

TABLE 11.—ELEVATIONS OF MEAN HIGH AND MEAN LOW WATER AT BOTH ENDS OF THE CANAL BEFORE AND AFTER OPENING TO FLOW.

	Buzzards Bay.		Cape Cod Bay.	
	High water.	Low water.	High water.	Low water.
Before opening	102.17	98.26	104.56	95.54
After opening	102.14	98.55	104.42	95.50

It will be seen that no appreciable influence was produced on the tidal elevations in Cape Cod Bay, nor on high-water elevation in Buzzards Bay, but that the elevation of low water in Buzzards Bay was raised 0.29 ft., or about 3½ in., due undoubtedly to the inability of the water to discharge itself freely in the shallow depth existing at low tide.

In order to ascertain all the conditions affecting, or produced by, the motion of water in the canal, an elaborate system of taking measurements was organized after the excavation had been completed so as to give free flow.

Observation posts were established at Stations 45, 80, 125, 172, 225, 275, 325, 375, 399+50, and 410. Station 45 is where the canal has a bottom width of 200 ft. At Station 80 the bottom width had been reduced to the normal 100 ft., the narrowing commencing at Station 70, Station 80 being selected for observation as being about the first or last place, according to direction of current, where the flow was believed to be normal for a narrow section. Through the 100-ft. section the observing stations were at about 5 000-ft. intervals to Station 399+50, where the cross-section is increased to 250 ft. bottom width. Station 410 is the end of the canal. At all these stations, tide boards were set up, and their elevations were carefully checked.

At each post there was an experienced observer with two assistants, and the observations were made simultaneously and continuously for nearly 15 hours, so as to cover fully a complete tidal cycle.

One of the assistants was in a boat supplied with floats, of which he placed one in the center of the canal every 15 min. about 600 ft. above (according to current) the observer. The second assistant was stationed 500 ft. above the observer sighting across the canal over marks set at right angles to the line of the canal. The accurately placed canal lights served well for such purpose. When the float passed the assistant he signaled to the observer who recorded the time of passage, and again the time when the float passed similar marks at the observing station. These times by reduction gave the center surface velocities. At 15-min. intervals the observer also recorded the elevation of the water, as shown by the tide boards, and noted the direction, time of stopping, and reversing of the current, action of the wind, passing of boats, and other circumstances affecting the flow. The observers' watches were synchronized by the engineer in charge. To eliminate errors these observations were repeated on July 26th and August 26th, 1914, being selected as days when a head differential greater than the mean was to be expected. All reductions and computations were made in the Chief Engineer's office from original notebooks, the observers being given no opportunity to check their recorded observations with their own figures or those of adjacent observers.

To permit these records to be visualized, they have been plotted in a series of diagrams.

The first of the series, Fig. 18, shows the simultaneous elevations of the water surface as taken at several observation stations. By connecting the elevations at the same time points by straight lines between observation stations, instantaneous profiles of water surface are obtained. These lines are not straight from one end of the canal to the other, but are substantially straight only for those portions of the canal where the cross-section is uniform, that is, between Stations 80 and 375, where the canal has a bottom width of 100 ft. The lines between Stations 45 and 80 are flatter on account of the greatly increased cross-section of the canal. The slight irregularities occurring between Stations 375 and 380 are due to the contraction of the Buzzards Bay Bridge.

If curves are drawn osculatory to these lines, they will give the elevations of high water and low water at all points through the canal. The greater the number of simultaneous water profiles, the greater

will be the number of points given on the curves, but the more accurately determined loci of the curves will not vary appreciably from the curves as drawn on Figs. 18 and 19. These curves show that the elevations of high and low water are not on straight lines between the two ends of the canal, as the surface slopes are, but lie on pronounced curves; and that, for substantial portions of the length of the canal, tides rise neither as high nor fall as low even as the high and low minimum of Buzzards Bay. This result was not anticipated, nor was it suggested by any writer on the expected tidal results. The differences are quite material, as shown by Table 12, which gives the actual elevations at points in the canal and computed elevations if the high and low water were on straight lines, connecting the high and low points at Buzzards Bay and Cape Cod Bay.

TABLE 12.—DEVIATION OF ACTUAL HIGH-WATER AND LOW-WATER LINES FROM STRAIGHT LINES CONNECTING THE EXTREME ELEVATIONS AT THE TWO ENDS OF THE CANAL.

Stations.	High Water.			Low Water.		
	Actual elevation.	Elevation of straight line.	Difference, in feet.	Actual elevation.	Elevation of straight line.	Difference in feet.
Observations of July 26th, 1916.						
35 + 00	104.80	104.8	....	93.87	93.87	....
45 + 00	104.96	104.77	+ 0.19	93.87	93.98	+ 0.11
80 + 00	104.60	104.66	− 0.06	94.23	94.40	+ 0.17
125 + 00	104.16	104.52	− 0.36	94.70	94.93	+ 0.23
172 + 00	103.50	104.37	− 0.87	95.70	95.48	− 0.22
225 + 00	102.80	104.20	− 1.40	96.90	96.09	− 0.81
275 + 00	102.33	104.04	− 1.71	97.60	96.67	− 0.93
325 + 00	102.75	103.88	− 1.13	98.16	97.25	− 0.91
374 + 20	103.25	103.70	− 0.45	98.21	97.80	− 0.59
380 + 00	103.33	103.67	− 0.34	98.16	97.87	− 0.29
399 + 50	103.66	103.66	....	98.00	98.00	....
Observations of August 26th, 1916.						
35 + 00	105.58	105.58	....	94.75	94.75	....
45 + 00	105.62	105.53	+ 0.09	94.72	94.84	+ 0.12
80 + 00	105.50	105.30	+ 0.20	94.50	95.16	+ 0.66
125 + 00	104.80	105.02	− 0.22	95.70	95.59	− 0.11
172 + 00	104.13	104.73	− 0.60	96.20	96.05	− 0.15
225 + 00	103.30	104.46	− 1.16	96.90	96.45	− 0.45
275 + 00	102.50	104.15	− 1.65	97.55	96.94	− 0.61
325 + 00	102.50	103.85	− 1.35	98.30	97.42	− 0.88
375 + 00	103.04	103.53	− 0.49	98.65	97.90	− 0.75
380 + 00	103.10	103.50	− 0.40	98.60	97.95	− 0.65
410 + 00	103.30	103.30	....	98.21	98.21	....

Differences are marked with the minus sign for actual elevations falling below the straight line for high water, and for elevations falling above the straight line for low water.

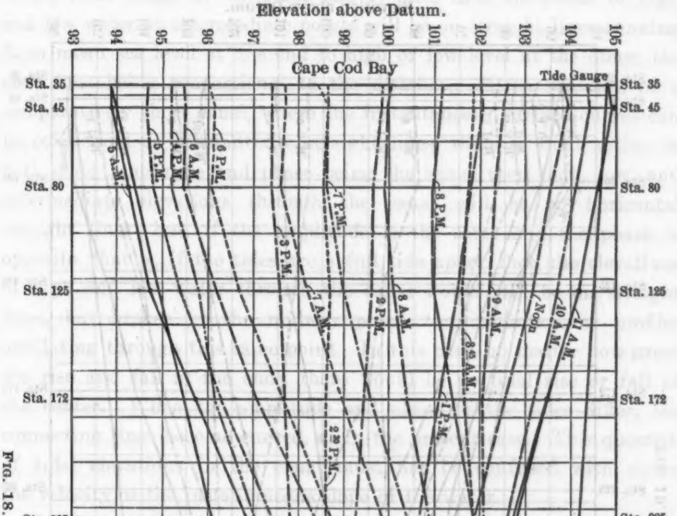

Elevation above Datum.
93
94
95
96
97
98
99
100
101
102
103
104
105
Cape Cod Bay
Tide Gauge
Sta. 35
Sta. 45
Sta. 80
Sta. 125
Sta. 172
Sta. 225
Sta. 275
Sta. 325
Sta. 343
Sta. 374.2
Sta. 380
Sta. 399.5
Buzzards Bay
High Water Curve
Low Water Curve
Noon
DIAGRAM SHOWING WATER SURFACE OF
CAPE COD CANAL FOR EVERY HOUR
DURING OBSERVATIONS ON JULY 26th, 1915.
Points marked thus • show actual maximum
and minimum elevations, respectively, at every
section observed.
Elevation above Datum.

FIG. 18.

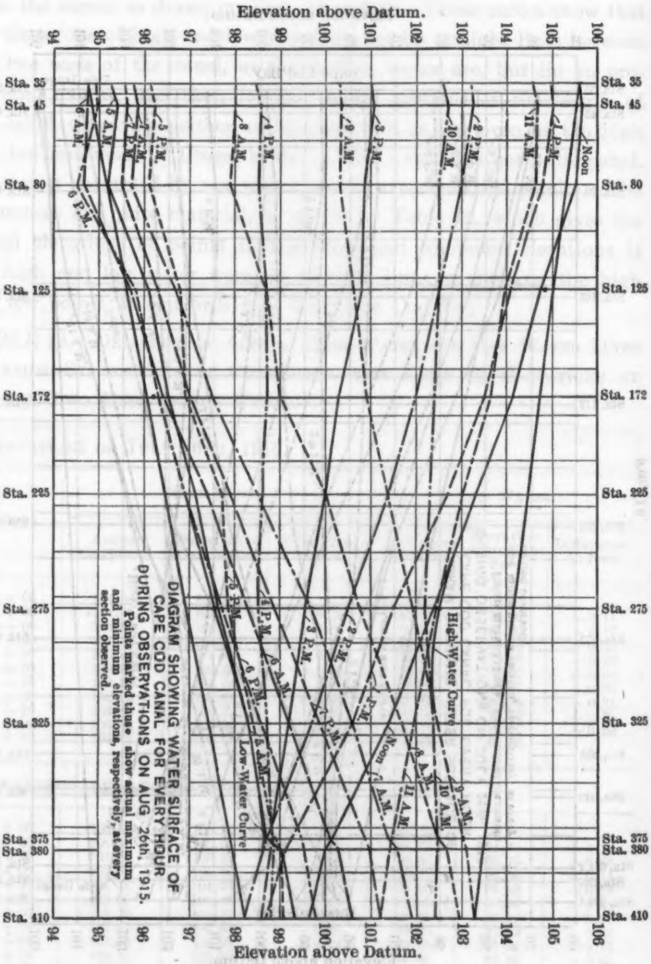

Elevation above Datum.
94
95
96
97
98
99
100
101
102
103
104
105
106
Sta. 35
Sta. 45
Sta. 80
Sta. 125
Sta. 172
Sta. 225
Sta. 275
Sta. 325
Sta. 375
Sta. 380
Sta. 410
Noon
11 A.M.
10 A.M.
9 A.M.
8 A.M.
6 A.M.
5 A.M.
1 P.M.
2 P.M.
3 P.M.
4 P.M.
5 P.M.
6 P.M.
High-Water Curve
Low-Water Curve
DIAGRAM SHOWING WATER SURFACE OF
CAPE COD CANAL FOR EVERY HOUR
DURING OBSERVATIONS ON AUG 26th, 1915.
Points marked thus show actual maximum
and minimum elevations, respectively, at every
section observed.
Elevation above Datum.

FIG. 19.

A moment's consideration will explain this curious phenomenon: If, in an open channel, there is tidal action at one end only, that is, if the tidal range at the other end is zero, then elevations of high and low water at intermediate points will be on straight lines running from mean sea level at one end to high or low level at the other, the elevations being proportional to the distance. If, in the case of a comparatively short canal, where the instantaneous surface curves can be considered as substantially straight lines, there is tidal action at both ends, amplitude and phase being the same, then high, low, and intermediate elevations through the canal will be on horizontal straight lines; but, if the amplitude is the same and the phase is opposite, that is, if the tides are a full tide apart, then the elevations of high and low water through the canal would still be on straight lines, but broken at the midway point, the instantaneous profiles oscillating through this same point. In this case, no matter how great the rise and fall at the ends, there would be no tidal rise or fall at the center. When the amplitude and phase of the tides differ, the connecting lines become curved, as in the present case. This question of tidal elevations in the canal must not be confused with either the velocity or the instantaneous slope of the water.

In Fig. 18 the curves of high and low water are not symmetrical, which at first thought appears should be the case. The lack of symmetry is due to the incompleteness of the lower branch of the Buzzards Bay tide curve. If the tide fell below mean sea level as much as it rises above, the apex of the low-water curve would be advanced so as to lie beneath the apex of the high-water curve.

The elevation and location of the apices of the high- and low-water curves are functions of the phase difference and relative magnitudes of the terminal tides.

The line of elevations of low water is of great practical importance in construction. In order to give a uniform depth of water at low tide, it is not necessary to give a uniform slope to the bottom. The line of low-water elevations should be determined, which can easily be predicated from tidal records at the ends, and the excavation be made parallel thereto. An inspection of the figures in Table 12 indicates the variation from computed straight-line or uniform-slope elevations, and the corresponding saving in unnecessary excavation can be estimated.

The current velocities ascertained by the observations were the velocities on the surface at the center of the canal, and no other method of measuring velocities was practicable, in view of the required great frequency of the observations at so many stations, subject as they were to interruption by passing vessels. In order to harmonize the observations with flow formulas which depend on and give mean velocity, it was necessary to ascertain the relation existing between the center surface velocity and the mean velocity of the whole cross-section, as it actually existed in a channel of these dimensions and with frictional resistance produced by the material forming the sides and bottom.

The method adopted to determine the mean velocity and its ratio to the center surface velocity was that recommended by the U. S. Coast and Geodetic Survey Bureau. A point in the canal, Station 225, was selected where, for a considerable distance in both directions, the alignment was a tangent and the cross-sections of the canal were substantially uniform. The various threads of flow, therefore, were straight, parallel, and practically undisturbed by local eddies.

A wire was stretched across the canal, clear of the water, and on it were fastened tags at intervals of 10 ft. A boat, with the measuring crew, was held at each tag by an anchor while velocity readings were made, beginning at the surface and then downward at intervals of 2 ft. Measurements were thus made on a spacing of 10 ft. horizontally and 2 ft. vertically from shore to shore and from surface to bottom.

The gauging was done with a Gurley-Price current meter, which had been accurately rated by the manufacturers immediately prior to use. The revolutions of the meter were counted for 30 sec., and the recording of the velocities in each full vertical took from 10 to 20 min. each. In addition to recording the readings of the meter, all attending circumstances were noted, such as the elevation of the water, direction and strength of the wind, passing of vessels, etc., and float velocity determinations were made as a check on the meter and the work, and especially to record the variations in the surface velocity during the measurements.

The gauging was begun on the center line on July 10th, 1915, at 9.35 A. M. and proceeded toward the north bank, the vertical, 90 north, being measured at 11.51 A. M., when the elevation of the water surface had fallen from 101.7 to 100.5. Beginning again at vertical,

10 south, at 11.56 A. M., vertical, 90 south, was finished at 1.43 P. M., when the water surface stood at Elevation 99.6. During the latter half of the work the current had slackened considerably, so that the readings in the south half of the canal section were much lower than at corresponding points in the north half. In order to correct this discrepancy, readings in the south half were repeated 4 days later when local conditions of tide, current, and wind were comparably similar to those prevailing on the first occasion.

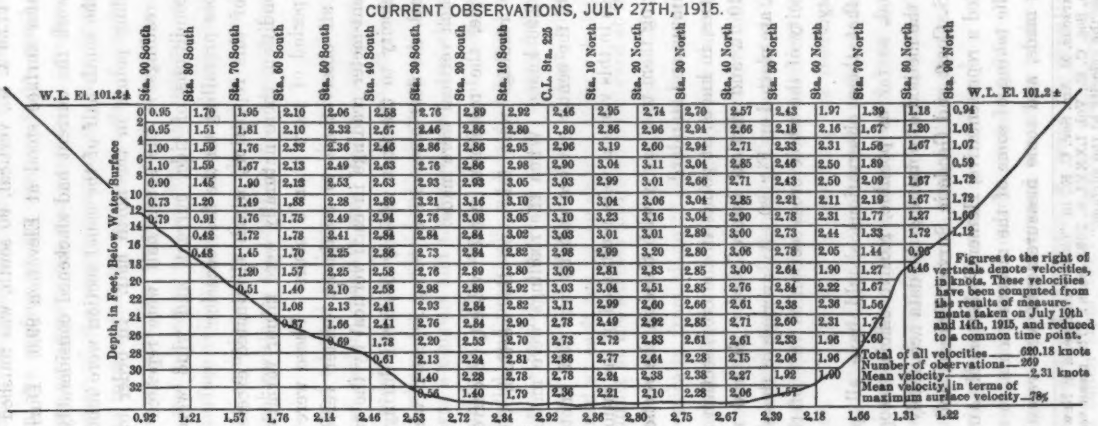
In spite of this repetition, which eliminated excessive variation in current condition, there naturally were minor variations, as developed over a period of more than 2 hours. These were compensated by reducing all observations to maximum center velocity by multiplying current-meter readings in each vertical by the ratio of maximum float velocity to the float velocity observed during the reading at such vertical, the float observations being repeated when the meter readings on each vertical were made.

For example, the maximum center velocity occurred when the current-meter readings were taken on the vertical, 30 north, and amounted to 2.96 knots. When the readings were made on the vertical, 60 north, the center surface velocity was 2.55 knots, and therefore all readings in this vertical were multiplied by the ratio, $\frac{2.96}{2.55} = 1.16$ in order to bring them to a parity with those in the vertical, 20, which were the maximum and standard.

The velocities, in knots, at points in the cross-section at horizontal intervals of 10 ft. and vertical intervals of 2 ft. and reduced to a common basis, are platted on Fig. 20. An average of these figures gives the mean velocity of the whole cross-section as 78% of the maximum surface velocity.*

In order that these observations should have all personal bias eliminated, and, so far as possible, receive official recognition, suggestions as to the method of obtaining the data accurately were invited from the U. S. Coast and Geodetic Survey Bureau, and that Bureau kindly delegated a representative, Homer P. Ritter, M. Am. Soc. C. E., to supervise the taking of some of the measurements, to see that they were properly made, and some measurements were repeated in order

* H. de B. Parsons, M. Am. Soc. C. E., in "Tidal Phenomena in New York Harbor," *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 2032, says: "The mean sectional velocity is about 0.75 times the velocity at the surface."



Depth	Sta. 90 South	Sta. 80 South	Sta. 70 South	Sta. 60 South	Sta. 50 South	Sta. 40 South	Sta. 30 South	Sta. 20 South	Sta. 10 South	C.L. Sta. 225	Sta. 10 North	Sta. 20 North	Sta. 30 North	Sta. 40 North	Sta. 50 North	Sta. 60 North	Sta. 70 North	Sta. 80 North	Sta. 90 North
0	0.95	1.70	1.95	2.10	2.06	2.58	2.76	2.89	2.92	2.46	2.95	2.74	2.70	2.57	2.43	1.97	1.39	1.18	0.94
2	0.95	1.51	1.81	2.10	2.32	2.67	2.46	2.86	2.80	2.80	2.86	2.96	2.94	2.66	2.18	2.16	1.67	1.20	1.01
4	1.00	1.59	1.76	2.32	2.36	2.46	2.86	2.86	2.95	2.96	3.19	2.60	2.94	2.71	2.33	2.31	1.56	1.67	1.07
6	1.10	1.59	1.67	2.13	2.49	2.63	2.76	2.86	2.98	2.90	3.04	3.11	3.04	2.85	2.46	2.50	1.89		0.59
8	0.90	1.45	1.90	2.13	2.53	2.63	2.93	2.93	3.05	3.03	2.99	3.01	2.66	2.71	2.43	2.50	2.09	1.67	1.72
10	0.73	1.20	1.49	1.88	2.28	2.89	3.21	3.16	3.10	3.10	3.04	3.06	3.04	2.85	2.21	2.11	2.19	1.67	1.72
12	0.79	0.91	1.76	1.75	2.49	2.94	2.76	3.08	3.05	3.10	3.23	3.16	3.04	2.90	2.78	2.31	1.77	1.27	1.60
14		0.42	1.72	1.78	2.41	2.84	2.84	2.84	3.02	3.03	3.01	3.01	2.89	3.00	2.73	2.44	1.33	1.72	1.12
16		0.48	1.45	1.70	2.25	2.86	2.73	2.84	2.82	2.98	2.99	3.20	2.80	3.06	2.78	2.05	1.44	0.96	
18			1.20	1.57	2.25	2.58	2.76	2.89	2.80	3.09	2.81	2.83	2.85	3.00	2.64	1.90	1.27	0.46	
20			0.51	1.40	2.10	2.58	2.98	2.89	2.92	3.03	3.04	2.51	2.85	2.76	2.84	2.22	1.67		
22				1.08	2.13	2.41	2.93	2.84	2.82	3.11	2.99	2.60	2.66	2.61	2.38	2.36	1.56		
24				0.87	1.66	2.41	2.76	2.84	2.90	2.78	2.49	2.92	2.85	2.71	2.60	2.31	1.77		
26					0.69	1.78	2.20	2.53	2.70	2.73	2.72	2.83	2.61	2.61	2.33	1.96	1.60		
28						0.61	2.13	2.24	2.81	2.86	2.77	2.64	2.28	2.15	2.06	1.96			
30							1.40	2.28	2.78	2.78	2.24	2.38	2.38	2.27	1.92	1.90			
32							0.56	1.40	1.79	2.36	2.21	2.10	2.28	2.06	1.57				
Mean velocities in verticals, in knots	0.92	1.21	1.57	1.76	2.14	2.46	2.53	2.72	2.84	2.92	2.86	2.80	2.75	2.67	2.39	2.18	1.66	1.31	1.22

FIG. 20.

to check the accuracy of those previously recorded. The Engineer of the Harbor and Land Commission of Massachusetts did likewise.

An inspection of Fig. 20 shows that the maximum velocity, as in other streams, is not at the surface, but at a considerable distance below it, and that the velocity increases from that at the surface to the maximum and then decreases with further increase in depth. The measured velocities in the seven central verticals, 30 south to 30 north, both inclusive, in which the velocities are the maximum, and which are the least affected by side resistance and eddies, have been averaged and platted in the solid line on Fig. 21.

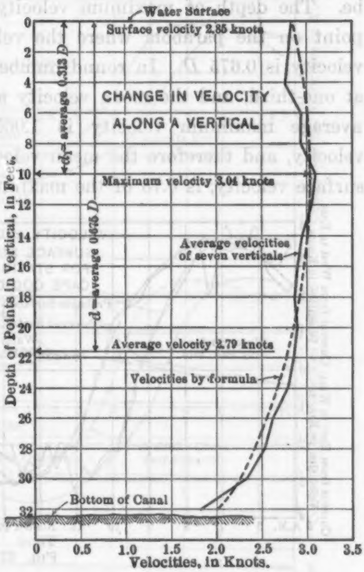


FIG. 21.

An analytical expression of the change in vertical velocity in a stream is given by the parabolic formula proposed by Capt. Humphreys and Lieut. Abbot, Topographical Engineers, U. S. A., in their memorable classic "The Hydraulics of the Mississippi River" in 1861:

$$V = V_{d_1} - (b\,v)^{\frac{1}{2}} \left(\frac{d - d_1}{D}\right)^2$$

where V = velocity at a point in the vertical the depth of which below the surface is denoted by d;

V_{d_1} = maximum velocity in the vertical;

v = mean velocity of the whole cross-section;

D = depth of vertical;

d_1 = depth of the point of maximum velocity in the vertical below the surface;

d = depth of any point in the vertical below the surface;

$b = \dfrac{\text{constant}}{\sqrt{D} + 1.5}$, the value of which for the section of the canal under consideration was found to be 2.93.

The curve given by this formula is shown by the dotted line on Fig. 22, its point of origin at the surface being taken coincident with the measured surface velocity. The measured vertical velocity line is, as was to be expected, a broken and not a regular curve, but it corresponds with astonishing closeness with the theoretical parabola, so closely that the latter is a fair average of the former, as it should be. The depth of maximum velocity is 0.313 *D*. The depth to the point on the parabola where the velocity is the same as the mean velocity is 0.675 *D*. In round numbers, the maximum velocity occurs at one-third, and the mean velocity at two-thirds, of the depth. The average maximum velocity is 1.066 times the maximum surface velocity, and therefore the mean velocity, being 0.78 of the maximum surface velocity, is 0.73 of the maximum velocity.

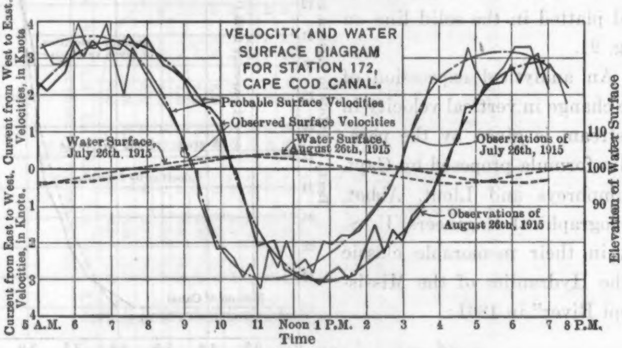


FIG. 22.

The plotting of the velocity observations gave curves which were not smooth but broken. These breaks in alignment were due to the inevitable variations in contiguous observations and to the fact that water in a large channel does not flow with absolute uniformity but in pulsations developed from many causes, such as the roughness of the bottom and sides, producing eddies; the local action of wind, and the passing of boats setting up waves that are felt at considerable distances. These irregularities were disregarded, and a smooth curve in each case was plotted, which was the average of the observations. As a matter of interest, Fig. 22 shows the actual observations as made at Section 172 (a fair example) and the smooth curve which was taken as the basis for computation.

Figs. 23 to 27 show the smooth curves at each of the observation stations, the right-hand scale referring to center surface velocities, the left-hand scale to the velocities reduced to mean velocity for the whole section by multiplying the observed surface velocities by 0.78, the ascertained coefficient. The abscissas of these curves represent

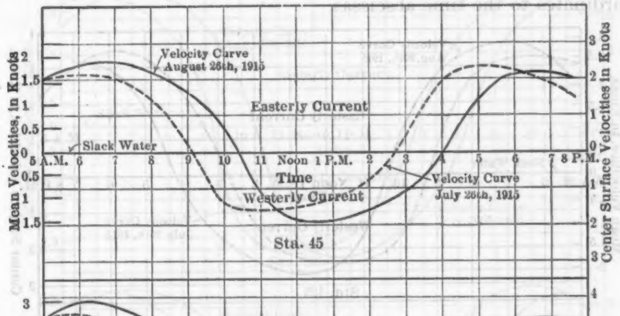


FIG. 23.

the time, and the velocities, in knots* per hour, are plotted as ordinates, above the axis for easterly and below for westerly current. Where the direction of the current is referred to as easterly or westerly, it is to be understood as being toward the east or west; that is, from Buzzards Bay to Cape Cod Bay or the reverse, respectively. The

*1 knot = 1 nautical mile (6 080 ft.), per hour, = 1 knot per hour = 1.69 ft. per sec.

observations on both occasions (July 26th and August 26th) are shown with evident closeness in results.

To show the relation existing between velocity and "head", curves are plotted at the bottom of each figure showing the measured differences in water elevations at the two ends of the canal, also plotted as ordinates to the time abscissas.

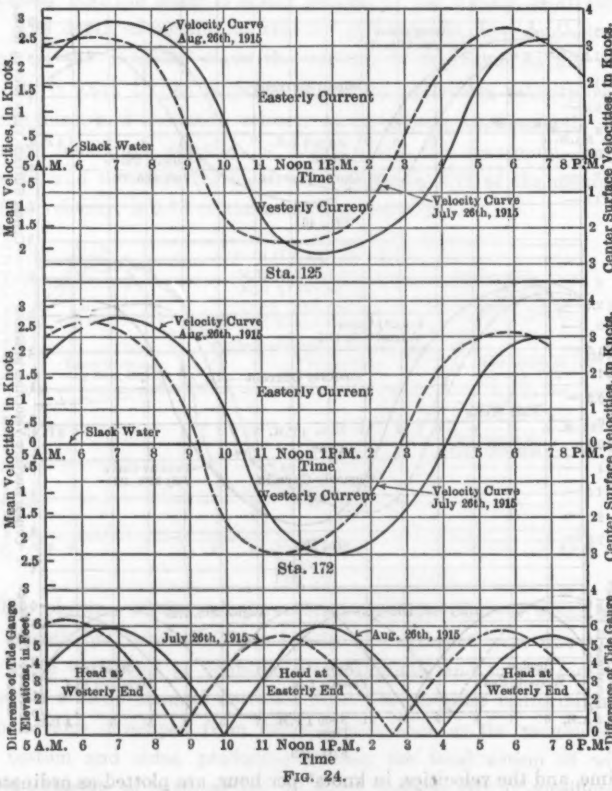


Fig. 24.

The striking and important characteristic features of these curves are:

1.—They closely resemble each other as to maxima and shape, thus providing a check on the accuracy of the observers. The variation in maximum readings, except for Stations 45, 375, 399+50, and

410, where the canal cross-section is greatly increased, and at Station 80 on the easterly and Station 375 on the westerly current, to be referred to later, are readily accounted for by slight variations in local conditions.

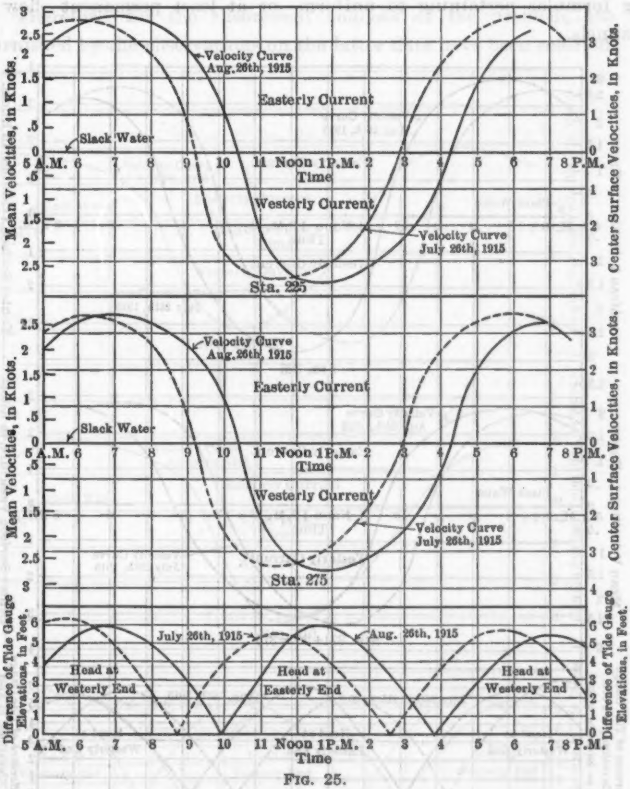


FIG. 25.

2.—The form of the curve approximates that of a sine curve, that is, the rate of change of the absolute value of the velocities is zero at the maximum in both directions of flow, and increases gradually toward zero velocities. The change of sign takes place very rapidly at the reversal of the current. At maxima velocities the character of flow approximates that of uniform motion.

3.—Maximum and zero velocities occur at very nearly the same instants throughout the whole length of the canal. These last two features indicate that, for the special case of the Cape Cod Canal, good approximate values for maxima velocities can be derived by applying formulas pertaining to uniform—or at least permanent—flow in channels.

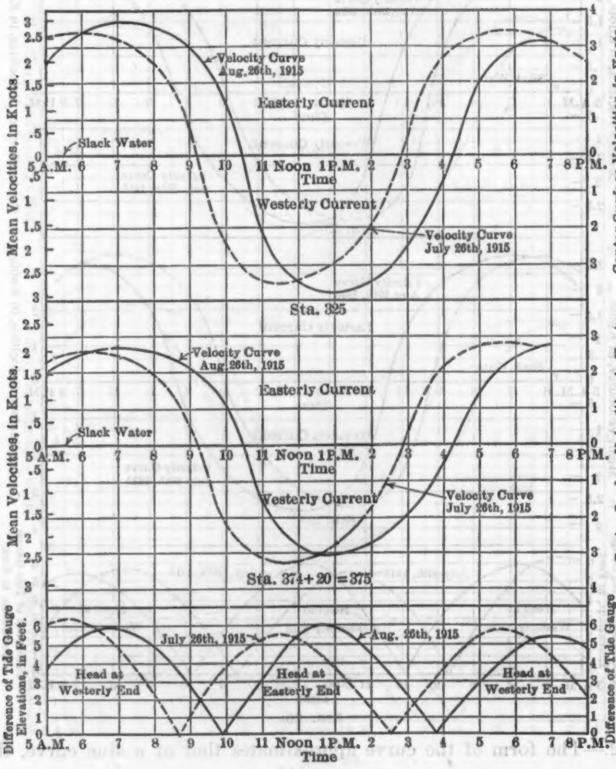


FIG. 26.

4.—The descending branches of the velocity curves have a somewhat greater inclination than that of the ascending branches.

5.—The duration of the easterly current is sensibly longer than that of the westerly current.

The diagram of elevation differences shows that though, for some local cause, the maximum head was considerably lower for westerly than for easterly currents during the observations on July 26th, on August 26th the maxima heads in opposite directions were almost equal. Therefore, for the theoretical analysis of the problem, the data furnished by the observations on the latter date have been selected.

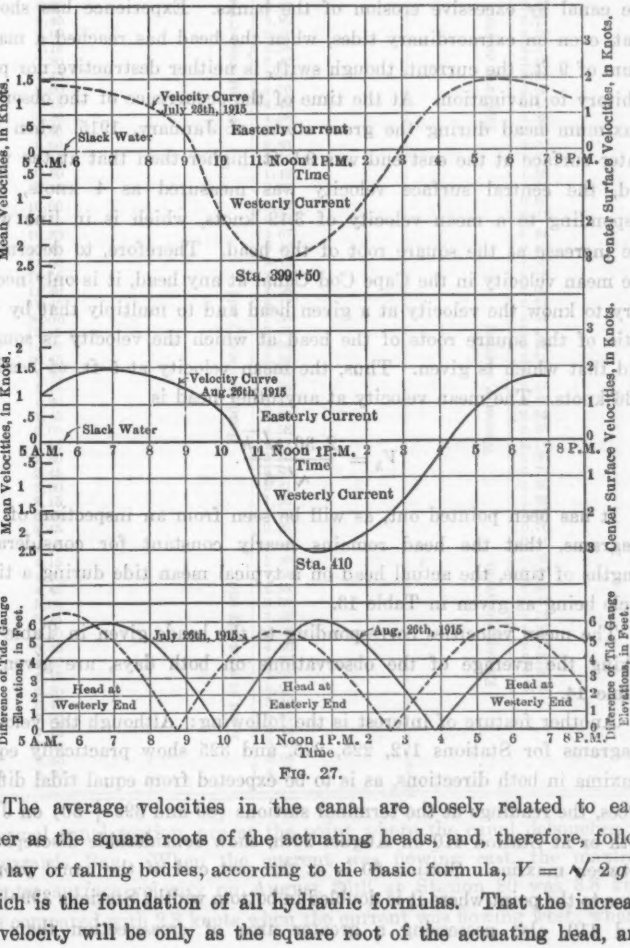


FIG. 27.

The average velocities in the canal are closely related to each other as the square roots of the actuating heads, and, therefore, follow the law of falling bodies, according to the basic formula, $V = \sqrt{2gh}$, which is the foundation of all hydraulic formulas. That the increase in velocity will be only as the square root of the actuating head, and

not as the full head, was a fact overlooked by many in considering the possibility of opening a canal at sea level without locks. Although admitting the possibility of success for the Cape Cod Canal at heads of 5 ft., some people had serious doubts as to what would happen at times of great storms piling up the water at one end and depressing it at the other, their fears extending even to the complete destruction of the canal by excessive erosion of the banks. Experience has shown that, even on extraordinary tides, when the head has reached a maximum of 9 ft., the current, though swift, is neither destructive nor prohibitory to navigation. At the time of the occurrence of the observed maximum head during the great storm of January, 1915, when the water surface at the east end was 9.5 ft. higher than that at the west end, the central surface velocity was measured as 4 knots, corresponding to a mean velocity of 3.12 knots, which is in line with the increase as the square root of the head. Therefore, to determine the mean velocity in the Cape Cod Canal at any head, it is only necessary to know the velocity at a given head and to multiply that by the ratio of the square roots of the head at which the velocity is sought and that which is given. Thus, the mean velocity at 5 ft. of head is 2.36 knots. The mean velocity at any other head is

$$V_h = \frac{2.36 \sqrt{h}}{\sqrt{5}}.$$

It has been pointed out, as will be seen from an inspection of the diagrams, that the head remains nearly constant for considerable lengths of time, the actual head on a typical mean tide during a tidal cycle being as given in Table 13.

The mean velocities corresponding to the heads given in Table 13, taking the average of the observations on both days, are given in Table 14.

Another feature of interest is the following: Although the velocity diagrams for Stations 172, 225, 275, and 325 show practically equal maxima in both directions, as is to be expected from equal tidal differences, the readings at the terminal stations (80 and 399 + 50) on July 26th or at Station 410 on August 26th, show considerable discrepancy between maxima. Station 80 is at the east end of the canal proper, close to the point where it is doubled in bottom width; Stations 399 + 50 and 410, also possessing a greater area of cross-section than the

TABLE 13.—TIDAL DIFFERENCES ON AUGUST 26TH, 1916.

Time, in hours and minutes.		Head, in feet.		Mean velocity in canal, in knots.
5.00 A. M.	Head at westerly end	3.75	Easterly current	2.03
5.15		4.35		2.21
5.30		4.75		2.35
5.45		5.17		2.48
6.00		5.50		2.57
6.15		5.83		2.64
6.30		5.87		2.70
6.45		5.89		2.72
7.00		5.72		2.72
7.15		5.58		2.71
7.30		5.42		2.68
7.45		5.10		2.63
8.00		4.70		2.55
8.15		4.42		2.45
8.30		4.08		2.33
8.45		3.46		2.19
9.00		2.83		2.02
9.15		2.15		1.83
9.30		1.42		1.58
9.45		0.36		1.27
10.00	Head at easterly end	0.12		0.89
10.15		0.92		0.41
10.30		1.67	Westerly current	0.24
10.45		2.42		0.87
11.00		3.12		1.39
11.15		3.96		1.76
11.30		4.58		2.02
11.45		5.17		2.21
Noon		5.58		2.34
12.15 P. M		5.83		2.43
12.30		5.96		2.49
12.45		5.92		2.51
1.00		5.88		2.51
1.15		5.66		2.47
1.30		5.42		2.42
1.45		5.08		2.34
2.00		4.58		2.23
2.15		4.12		2.11
2.30		3.50		1.95
2.45		2.83		1.76
3.00		2.21		1.56
3.15		1.66		1.32
3.30		1.00		1.05
3.45		0.25		0.76
4.00	Head at westerly end	0.42		0.41
4.15		1.00	Easterly current	0.02
4.30		1.62		0.55
4.45		2.33		1.08
5.00		2.92		1.47
5.15		3.42		1.74
5.30		3.92		1.97
5.45		4.30		2.14
6.00		4.58		2.27
6.15		4.95		2.36
6.30		5.25		2.43
6.45		5.42		
7.00		5.42		
7.15		5.42		
7.30		5.30		
7.45		5.00		
8.00		4.75		

normal canal section, are at the point where the canal debouches into Buzzards Bay. When the current was flowing east, the maximum center surface velocity on August 26th at Station 80 was 3.8 knots, as compared with 2.8 knots when the current was flowing west; whereas,

TABLE 14.

Head, in feet.	Observed mean velocity in canal, in knots.	Mean velocity by formula, $2.36 \times \frac{\sqrt{h}}{\sqrt{5}}$ knots.
3	1.69	1.83
3½	1.93	1.98
4	2.08	2.12
4½	2.20	2.24
5	2.36	2.36
5½	2.50	2.48
6	2.62	2.60

at Station 225, on the same date, the maximum center surface velocity was 3.7 knots in each direction. At the west end of the canal at Station 410 the maximum center velocity on the east-bound current was less than 2 knots and slightly more than 3 knots on the west-bound current.

That is to say, in both cases when the current was flowing from a narrow to a broader cross-section, the center surface velocity was increased; and it was decreased when the flow was in the opposite direction, or from broad to narrower. As the volume of water passing Station 80 is the same as in any other normal canal cross-section, though the center velocity is quite different, it must be that the same relation between center surface velocity and mean velocity does not exist.

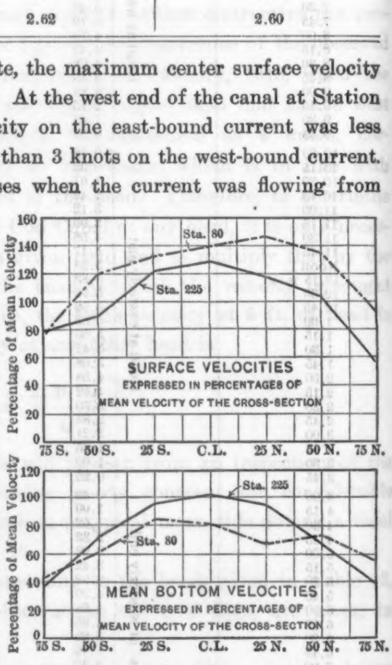


FIG. 28.

To prove this assumption, current-meter measurements were made at Stations 75 and 85, and the average of these showed that the mean velocity of the cross-section at Station 80 is only about 68%, instead of 78%, of the center surface velocity when the current is flowing in an easterly direction. In Fig. 28 the velocities on the surface and near the bottom of the canal are plotted, expressed in percentages of the mean velocity for the whole cross-section for Stations 80 and 225, for

comparison. This diagram shows that though the surface velocities at Station 80 are measurably greater than those at Station 225, the velocities near the bottom are in the inverse ratio. No measurements were made on the westerly flow, but undoubtedly the opposite conditions exist; that is, there is a higher proportionate bottom and side velocity to balance the known lower surface velocity, so that the mean cross-sectional velocity would have a higher ratio to the center surface velocity than the normal figure of 0.78, as that on the easterly flow is lower, the ratio being about 0.85 to 0.88. As this phenomenon is repeated at both ends of the canal, it is evidently not a matter of chance, and an explanation is offered that, when the current is from a narrow to a broad section, there is a contraction similar in character to that which occurs when a jet issues from an orifice; but when the current is from broad to narrow the diverted threads of flow at the sides, which find no place in the narrow section, if continued in straight lines, are pushed toward the center, and increase the velocity of flow at the sides, thus automatically reducing the velocity of the surface center flow, as the volume of water passing the section in a given interval of time is the same; and if the velocity at any part of the cross-section is increased, a compensative reduction must take place at some other point.

In a canal, therefore, the transition to increased cross-section should be made very gradually. In the canal in question a length of 500 ft. was given to a gradual widening, but this is seen to be insufficient to eliminate all variations in flow.

A phenomenon of much interest, as shown by these diagrams, is that the time of neither maximum current nor zero current coincides with the time of maximum tidal difference or of simultaneous equal end elevation.

Averaging the times of maximum and zero currents at Stations 80 to 375, both inclusive, so as to eliminate local variations in observations, the lag in time for the foregoing current conditions is:

Maximum easterly current behind Buzzards Bay high. .0.31 hour.
Maximum westerly current behind Cape Cod Bay high. .0.21 "
Zero velocity current behind equal elevation, Buzzards Bay tide falling ....0.51 "
Zero velocity current behind equal elevation, Cape Cod Bay tide falling ....0.44 "

By "Buzzards Bay tide falling" or "Cape Cod Bay tide falling" is meant that the tide at the end named is ebbing, and what had been superelevation at that end immediately prior to the moment of equal end elevation is about to be reversed.

It will be noted that the lag of zero velocity in both cases is greater than the lag of maximum velocity, and that the lag of the maximum or minimum (= 0) state of the current when governed by Cape Cod tidal influence is greater than when governed by Buzzards Bay tidal influence. An examination of the current diagrams shows in all cases a steeper inclination to the descending than to the ascending part of the curve, which accounts for the difference in lag.

A singular outcome of this phenomenon is that, with equal end elevations following Buzzards Bay falling tide, there is a current with an average mean velocity of more than 1 knot per hour, although at that instant there is no head to produce it. When the current velocity is zero (or slack water), there is a difference in head in Cape Cod Bay over Buzzards Bay of 1.5 ft. Between those limits of time (30 min.) the water in the canal is actually running up hill, from a maximum current of 1.1 knots when the slope is level to a current of zero velocity when the adverse head is 1.5 ft. On the other tide, when the end elevations are equal, the mean current velocity is 0.74 knot, and when the velocity is zero the adverse head, opposite to what has been producing flow immediately previous, is 1.0 ft.

The explanation of these phenomena of lag lies in the dynamic properties of moving liquids. A time interval is required to develop full momentum imparted by an extraneous force, in this case the full momentum not being reached until after its creating force has passed the apex of its energy. In like manner, momentum when once set up continues unaided until absorbed by friction or checked by a new and opposing force. The case of an inflowing tide in a narrow estuary being stopped by a dam or lock and producing at that point a higher elevation to high tide and a lower one to low tide than the normal is well known. In the Cape Cod Canal there is no such abrupt stop, but there is seen a very beautiful illustration of the balancing and oscillating action of two waves in their alternate development and arresting of motion. The rising tide at one end, when its elevation exceeds the tide at the other, produces a force in opposed head to overcome the momentum imparted by the previous "head" at the other end, and

then it in turn establishes a return flow. It will be noted that, when the velocity at 0 ft. differential of elevation is 1.1 knots, an increasing head to a maximum of 1.5 ft. is necessary to check it, and, when it has the opposite direction of velocity of 0.74 knot, a head of 1.0 ft. suffices. On account of this lag, the diagrams must be read with care to determine the velocity corresponding to any given head. A direct projection from the "head" to the "velocity" curve will not give the accurate rate; allowance must be made for lag. The harmonic analysis of this and other features of the problem is presented in the mathematical consideration.

A further peculiarity of the motion of the water in the Cape Cod Canal is revealed by inspecting Fig. 19, which represents the instantaneous surface curves of the canal for August 26th, 1915. It will be noted that at Station 172 the inclination of the surface had exactly the same value at noon (westerly current) as at 6.00 P. M. (easterly current), but the elevations of the water were 104.13 and 96.30, respectively, the hydraulic radius being 22.25 ft. at noon and 17.95 ft. at 6.00 P. M. From the principles of hydraulics, therefore, the velocity of the flow at noon should have been about 16% greater than at 6.00 P. M. $\left(\frac{v_1}{v_2} = \frac{C_1 \sqrt{R_1 S}}{C_2 \sqrt{R_2 S}} = 1.16\right)$. However, referring to the corresponding velocity diagram, it will be found that the velocities were almost exactly the same at these two time points. A similar result is obtained for Station 225, comparing observations at 7.00 A. M. and 1.00 P. M. This fact indicates that a change of the hydraulic radius, resulting solely from the tidal variations of the depth, has only a slight influence on the velocities, and that the flow in tidal streams similar to the Cape Cod Canal obeys different laws than those governing the uniform motion of water in canals.

Another unique feature, the discussion of which is made possible by the measurements of water elevations at the Cape Cod Canal, is the very complicated method of the wave propagation therein. In a canal subjected to tidal influences at one end only, the propagation of the wave is at a nearly constant rate, and its velocity can be expressed approximately by $w = \sqrt{g H}$. (This formula will be discussed later in the mathematical part of the paper.) Measurements in the Suez Canal

showed a very close agreement between the computed and observed values of the velocity of wave propagation.

At the Cape Cod Canal the conditions are entirely different, the value of the velocity of wave propagation being a function of the time interval between the high-water points of the two ends and also of the relative magnitude of the amplitudes of the two tides. It is evident that high water along the canal at the consecutive stations will be reached in the time interval between the high waters at Buzzards Bay and Cape Cod Bay. On August 26th this time interval was about 3 hours 20 min., high water at Station 410 occurring at 8.45 A. M. and at Station 35 at 12.05 P. M. The corresponding average velocity is 3.12 ft. per sec., or only about one-tenth of that furnished by the formula.

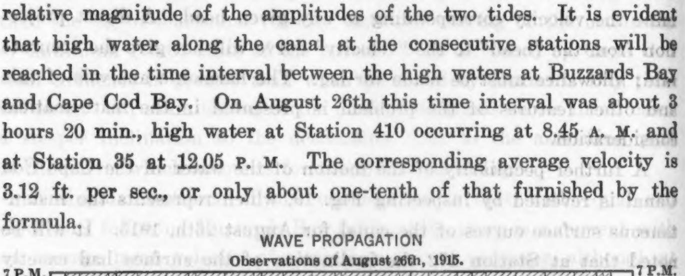


FIG. 29.

An inspection of Figs. 29 and 30, which show the time occurrence of high and low water and also of slack water along the canal, plotted as ordinates to the station abscissas, reveals the fact that the true propagation of the compound wave which results from the combination of the two tidal motions is very different from this average value. The propagation is very swift and sometimes almost instantaneous for the reaches near the ends of the canal, and it becomes very slow for about two-fifths of the length at high water and for about one-sixth at low

water. Figs. 29 and 30 show the overwhelming influence of the Cape Cod wave over the Buzzards Bay wave, the length of the corresponding nearly horizontal lines indicating time of high and low water along the canal being nearly proportional to the difference in amplitudes at these points. An interesting contrast to the propagation curve of high and low waters is the almost exactly simultaneous occurrence of slack water on the entire length of the canal, which is also shown on the diagrams.

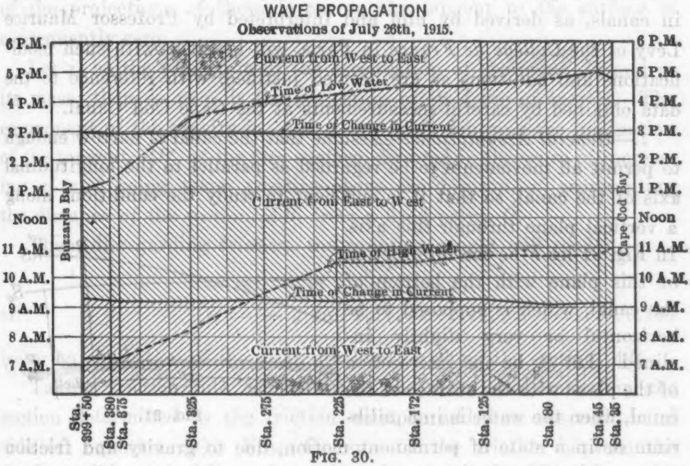


FIG. 30.

Although it does not seem possible to find an analytical expression for the velocity of the wave propagation in the Cape Cod Canal, the problem can be solved with the aid of the harmonic analysis of the motion, by calculating the water elevation curve for every station (as will be shown later) and picking out the high, low, and slack-water times of the same and plotting these times as ordinates to the station abscissas.

THE SOLUTION OF THE PROBLEM BY HARMONIC ANALYSIS.

It is a somewhat curious fact that textbooks on hydraulics in use in engineering practice do not give any information whatsoever on the analytical side of tidal phenomena, and that even standard books on hydrodynamics deal with the practical problem of tidal currents in a very incomplete way. For the study of tidal motion in canals, one must search the libraries for very rare scientific publications,

and, therefore, it is hoped that a complete discussion of the question will be welcomed by the members of the Society and the readers of the *Transactions* in general.

The classical solution of the problem of tidal currents in canals was derived by Sir George Biddell Airy, of Cambridge, later Astronomer Royal, Greenwich Observatory, in his dissertation on "Tides and Waves."* All subsequent treatment of the subject has been based on Airy's work, and in the following pages the theory of tides in canals, as derived by him and interpreted by Professor Maurice Lévy of the *Collège de France* in Paris, will be given with such modifications and additions as are deemed justified, with reference to the data obtained by careful measurements at the Cape Cod Canal.

1.—General Assumptions.—Assume that the canal is narrow enough to permit all movements to be regarded as parallel to the longitudinal axis of the canal, so that it is sufficient to study the conditions along a vertical plane through this axis.

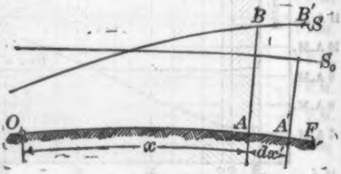


FIG. 31.

In Fig. 31 let F be the intersection of this plane with the bottom of the canal, which is supposed to be horizontal or very slightly inclined. Let S_0 be the intersection of the plane with the surface of the canal, when the water is in equilibrium or in a state of permanent motion, due to gravity and friction only, and let S be the intersection of the plane with the mobile surface, in case the motion of the water is not permanent, due to the action of the stars or any other cause.

Also assume that, at the same line, A-B, normal to the bottom, the longitudinal displacements will be practically the same for every particle. This has been proved by Airy, who in Section 180 says:

"When the length of the wave is great in comparison with the depth of the water (as in the case of tide waves), the horizontal motion is sensibly the same from the surface to the bottom * * *."

It is sufficient, therefore, to study the motion of the point, B, at the surface.

Assume that the tide generated within the canal is so small as to be negligible, and therefore the non-permanent motion, which it is proposed to investigate, is due to other direct causes.

* "Encyclopædia Metropolitana," Vol. V, 1845.

Finally, assume that the movements of the line, S, are small in relation to the depth of the canal.

2.—Differential Equations of the Varying Motion of the Water in Canals.—Hydrostatics teaches that for a fluid at rest the free surface is a surface normal to the resultant of the forces at each point of application, B. According to d'Alembert's theorem, the same holds good for the case of motion, but the force of inertia of the point considered should be added to the attacking forces. The algebraic sum of the projections of these forces on the tangent to the surface is consequently zero.

Let j_t be the projection of the acceleration of the point, B, and m its mass; then $-m\, j_t$ is the projection of its force of inertia. The component of the gravity is $-m\, g$ sin. I_s, I_s denoting the inclination of the tangent, BB', of the free surface, directed in the sense of increasing x' and being taken positive above the horizon, x' representing the distance of the section, AB, from a fixed point, O.

Then, disregarding friction, the equation sought will read

$$-m j_t - m\, g \text{ sin. } I_s = 0,$$

or

$$j_t = -g \text{ sin. } I_s \qquad \text{.........................(a)}$$

being an expression of the mean acceleration along the section, AB.

Taking friction into consideration, the mean acceleration in the section determined by the friction must be added to the right-hand side of Equation (a). Now, the sum of the friction between the filaments of water is zero, by virtue of the principle of action and reaction, and therefore only the friction of the water at the wetted perimeter should be considered. Let F be the friction per unit of wetted surface and X the wetted perimeter of the section, AB, and let the section, $A'B'$, be infinitely near to AB, at a distance, $x' + d\,x'$, from O. The wetted surface of the canal between the two sections equals $X\, d\, x'$, and the corresponding friction, $F\, X\, d\, x'$. The mass of liquid, AB-$A'B'$ is $\frac{\eta}{g}\, \Omega\, d\, x'$, denoting the area of the section, $A\,B$, by Ω, and the weight of the liquid per unit volume by η.

The mean acceleration due to the force, $F\, X\, dx'$, therefore, reads

$$\frac{F\, X\, d\, x'}{\frac{\eta}{g}\, \Omega\, d\, x'} = \frac{g\, F\, X}{\eta\, \Omega}$$

and, being directed opposite to the motion, Equation (a) becomes

$$j_t = - g \sin. I_s - \frac{g F X}{\eta \Omega} \dots\dots\dots\dots\dots (b)$$

Let y_f be the ordinate of the point, A, of the bottom of the canal, measured from an arbitrary datum line; this ordinate is a function of the one variable, x'; and let z be the depth of the canal at A at the instant, t, that is, z is a function of the two independent variables, x' and t. Then

$$\sin. I_s = \frac{\delta (y_f + z)}{\delta x'} = \frac{\delta y_f}{\delta x'} + \frac{\delta z}{\delta x'} = \sin. I + \frac{\delta z}{\delta x'} = I + \frac{\delta z}{\delta x'}$$

denoting the slope of the bottom by I, figured positive above the horizon, so that I is positive or negative according to whether, by ascending the bottom slope, we go in the direction of or against the positive, x'.

Assuming that the friction, F, per unit surface is proportional to the nth power of the velocity, v, so that

$$F = f_0 v^n \dots\dots\dots\dots\dots\dots\dots\dots (c)$$

f_0 being a coefficient varying with the consistency of the wetted perimeter and being equal to F for unit velocity, Equation (b) becomes

$$j_t = - g I - g \frac{\delta z}{\delta x'} - f v^n \dots\dots\dots\dots\dots\dots (d)$$

if, for the sake of simplicity, we put

$$f = \pm \frac{g f_0 X}{\eta \Omega} \dots\dots\dots\dots\dots\dots\dots\dots (e)$$

In Equation (a), f is positive always, if n is an odd figure, also, if n is even and $v > 0$; in other words, if the motion is in the direction of positive x'. If $v < 0$ and n is even, the lower sign should be taken.

In order to transform Equation (d) into an equation of partial derivatives, let us characterize a section by the abscissa, x, and the depth of the water at that section by γ at a particular instant, which is taken as the origin of time; the abscissa, x', and the depth, z, of the section at the instant, t, will then be functions of the two independent variables, t and x. To follow the motion of the particles of a section designated by its initial abscissa, x, it is sufficient to let the

time vary only. The expression for velocity and acceleration will consequently be

$$v = \frac{\delta x'}{\delta t}, \text{ and}$$

$$j_t = \frac{\delta v}{\delta t} = \frac{\delta^2 x'}{\delta t^2} \dots\dots\dots\dots\dots\dots\dots\dots (f)$$

Substituting in Equation (d)

$$\frac{\delta^2 x'}{\delta t^2} + f\left(\frac{\delta x'}{\delta t}\right)^n + g\, I = -\, g\, \frac{\delta z}{\delta x'},$$

and multiplying by $\frac{\delta x'}{\delta x}$, we get

$$\left[\frac{\delta^2 x'}{\delta t^2} + f\left(\frac{\delta x'}{\delta t}\right)^n + g\, I\right] \frac{\delta x'}{\delta x} = -\, g\, \frac{\delta z}{\delta x} \dots\dots\dots\dots (I)$$

which is one of the differential equations between the unknowns, x' and z.

The practical incompressibility of the water furnishes the other equation.

Ω being the area of the wetted cross-section at AB, let Ω_0 be the area of this same section at the initial instant. The volume of water included between this and an infinitely near section, having an abscissa, $x + d\,x$, is $\Omega_0\, d\,x$. At the instant, t, the abscissas of the two sections are x' and $x' + \frac{\delta x'}{\delta x} d\ x$, respectively. The volume included between them equals $\Omega\, \frac{\delta x'}{\delta x}\, d\,x$, which must be equal to $\Omega_0\, d\,x$, or

$$\Omega \frac{\delta x'}{\delta x} = \Omega_0 \dots\dots\dots\dots\dots\dots\dots\dots (II)$$

which is termed the equation of continuity.

3.—Application of the General Differential Equations to a Canal of Uniform Rectangular Cross-Section.—Let v_0 be the absolute value of the velocity in the canal for uniform motion, determined by the bottom slope and friction only. Then, for this condition,

$$x' = x - v_0 t, \text{ and } z = \gamma \dots\dots\dots\dots\dots\dots (f_a)^*$$

For the non-permanent oscillating motion,

$$x' = x - v_0 t + \xi, \text{ and } z = \gamma + h \dots\dots\dots\dots\dots (g)$$

where ξ and h represent the deviations of x' and z from the conditions of uniform motion, and therefore are assumed to be comparatively small.

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* The origin of the abscissas can always be selected in such a way that v_0 shall be negative.

Assume, further, that, for the range of velocities here considered, the power of the velocity, to which the friction is proportional, is equal to unity; then Equation (*I*) can be written

$$\left(\frac{\delta^2 x'}{\delta t^2} + f\,\frac{\delta x'}{\delta t} + g\,I\right)\frac{\delta x'}{\delta x} = -g\,\frac{\delta z}{\delta x} \qquad (h)$$

which, for permanent uniform flow, reduces to

$$-f\,v_0 + g\,I = 0 \qquad (i)$$

and substituting the value of $g\,I$ from Equation (*i*) in Equation (*h*), we get

$$\left[\frac{\delta^2 x'}{\delta t^2} + f\left(v_0 + \frac{\delta x'}{\delta t}\right)\right]\frac{\delta x'}{\delta x} = -g\,\frac{\delta z}{\delta x} \qquad (j)$$

Now, considering ξ and h as variables, as defined by Equation (*g*).

$$\frac{\delta^2 \xi}{\delta t^2} + f\,\frac{\delta \xi}{\delta t} = -g\,\frac{\frac{\delta h}{\delta x}}{1 + \frac{\delta \xi}{\delta x}} \qquad (k)$$

The cross-section being rectangular and uniform, $\Omega_0 = b_0\,\gamma$, and $\Omega = b_0\,z$, b_0, the width, being constant.

Therefore, from Equation (*II*),

$$\frac{z}{\gamma} = \frac{1}{\frac{\delta x'}{\delta x}} \qquad (l)$$

and again taking ξ and h as the variables, as defined by Equation (*g*), we get

$$z = \gamma + h = \frac{\gamma}{1 + \frac{\delta \xi}{\delta x'}} \qquad (m)$$

Now Equation (*k*), with respect to this last equation, can be written

$$\frac{\delta^2 \xi}{\delta t^2} + f\,\frac{\delta \xi}{\delta t} - g\,\gamma\,\frac{\frac{\delta^2 \xi}{\delta x^2}}{\left(1 + \frac{\delta \xi}{\delta x}\right)^3} = 0 \qquad (n)$$

and, neglecting the powers of $\frac{\delta \xi}{\delta x}$, as a first approximation, we finally have

$$\begin{cases} \dfrac{\delta^2 \xi}{\delta t^2} + f\,\dfrac{\delta \xi}{\delta t} - g\,\gamma\,\dfrac{\delta^2 \xi}{\delta x^2} = 0 \qquad (III)^* \\[2ex] h = -\gamma\,\dfrac{\delta \xi}{\delta x} \qquad (IV)^* \end{cases}$$

* See also Harris, "Manual of Tides," Part V, pp. 294 and 295.

It should be noted that these equations, although derived for a canal of rectangular cross-section, can be used, with the same degree of approximation, for uniform canals of any cross-section, supposing that by γ the "reduced depth" of the canal is understood, that is, the depth of a rectangular section having the same area and top width as the actual section. Substituting the value, $\Omega = \Omega_0 + b_0\, h$, and the value of x' from Equation (g) in Equation (II), we get

$$\Omega_0 \left(1 + \frac{\delta\, \xi}{\delta\, x}\right) + b_0\, h = \Omega_0.$$

Let $\Omega_0 = b\, \gamma$, γ being the reduced depth, as explained, then $\gamma \dfrac{\delta\, \xi}{\delta\, x} + h = 0$, which is identical with Equation (IV).

It can also be seen that the displacement caused by the uniform flow does not enter into the equation; such flow, therefore, if it exists, can be treated separately from the oscillating motion.

4.—General Integral of Equations (III) *and* (IV).—As the motion for which an expression is desired is only that which is periodical, there must be taken the most general expression, depending on the time, which will satisfy the Differential Equation (III). Assume then

$$\xi = P \cos.\, \sigma\, t + Q \sin.\, \sigma\, t \;\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

σ being an arbitrary constant, and P and Q functions of x to be discovered. Then

$$\frac{\delta\, \xi}{\delta\, t} = - \sigma P \sin.\, \sigma\, t + \sigma\, Q \cos.\, \sigma\, t$$

$$\frac{\delta^2\, \xi}{\delta\, t^2} = - \sigma^2 P \cos.\, \sigma\, t - \sigma^2\, Q \sin.\, \sigma\, t$$

$$\frac{\delta^2 \xi}{\delta\, x^2} = \frac{d^2 P}{d x^2} \cos.\, \sigma\, t + \frac{d^2\, Q}{d\, x^2} \sin.\, \sigma\, t$$

Substituting in Equation (III), we find

$$- \sigma^2\, P \cos.\, \sigma\, t - \sigma^2\, Q \sin.\, \sigma\, t - f\, \sigma\, P \sin.\, \sigma\, t + f\, \sigma\, Q \cos.\, \sigma\, t$$
$$- g\, \gamma \frac{d^2\, P}{d\, x^2} \cos.\, \sigma\, t - g\, \gamma \frac{d^2\, Q}{d x^2} \sin.\, \sigma\, t = 0.$$

This equation must be satisfied for all values of t. Therefore, for $t = 0$ and $\sigma\, t = \dfrac{\pi}{2}$, the following equations must hold, respectively

$$\left.\begin{aligned} - \sigma^2 P + f\, \sigma\, Q - g\, \gamma \frac{d^2 P}{d\, x^2} &= 0 \\ - \sigma^2 Q - f\, \sigma\, P - g\, \gamma \frac{d^2\, Q}{d\, x^2} &= 0 \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots (1_0)$$

Now let

$$P = A\,e^{\alpha x} \text{ and } Q = B\,e^{\alpha x} \ldots\ldots\ldots\ldots\ldots(2)$$

then, substituting in Equation (1_0) and putting $x = 0$, the constants, A, B, α, will satisfy the equations

$$\left.\begin{array}{l}(\sigma^2 + g\,\gamma\,\alpha^2)\,A - f\,\sigma\,B = 0\\ f\,\sigma\,A + (\sigma^2 + g\,\gamma\,\alpha^2)\,B = 0\end{array}\right\} \ldots\ldots\ldots\ldots(3)$$

Eliminating the ratio, $\frac{A}{B}$, between Equations (3), we get

$$(\sigma^2 + g\,\gamma\,\alpha^2)^2 + f^2\,\sigma^2 = 0 \ldots\ldots\ldots\ldots\ldots\ldots(3_0)$$

which furnishes the characteristic equation

$$\alpha^2 = \frac{-\sigma^2 \pm f\,\sigma\,\sqrt{-1}}{g\,\gamma} \ldots\ldots\ldots\ldots\ldots(4)$$

from which

$$\alpha = p + q\,\sqrt{-1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

and p and q are defined by

$$\left.\begin{array}{l}p^2 - q^2 = -\dfrac{\sigma^2}{g\,\gamma}\\[2ex] 2\,p\,q = \pm\dfrac{f\,\sigma}{g\,\gamma}\end{array}\right\} \ldots\ldots\ldots\ldots(6)$$

which give

$$\left.\begin{array}{l}p^2 = \dfrac{\sigma^2}{2\,g\,\gamma}\left[-1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}\right]\\[3ex] q^2 = \dfrac{\sigma^2}{2\,g\,\gamma}\left[\,1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}\right]\end{array}\right\} \ldots\ldots(7)$$

we also can write

$$\left.\begin{array}{l}p = \dfrac{\sigma}{\sqrt{2\,g\,\gamma}}\sqrt{-1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}}\\[3ex] q = \dfrac{\sigma}{\sqrt{2\,g\,\gamma}}\sqrt{1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}}\end{array}\right\} \ldots\ldots\ldots\ldots(8)$$

We get two solutions

$$\alpha = p + q\,\sqrt{-1} \text{ and } \alpha = -p + q\,\sqrt{-1} \ldots\ldots(9)$$

and two others by changing q to $-q$. The first value of α in Equation (9) gives $\alpha^2 = \dfrac{-\sigma^2 + f\,\sigma\,\sqrt{-1}}{g\,\gamma}$ and, with respect to Equation (3), $B = A\,\sqrt{-1}$. The second value of α in Equation (9) gives $\alpha^2 = \dfrac{-\sigma^2 - f\,\sigma\,\sqrt{-1}}{g\,\gamma}$, and, consequently, $B = -A\sqrt{-1}$. There-

fore, without regard to the constant, A, the first value of α in Equation (9) gives

$$P = e^{(p+q\sqrt{-1})x} \text{ and } Q = \sqrt{-1}\, e^{(p+q\sqrt{-1})x}$$

which can also be written

$$P = e^{px} (\cos. q\, x + \sqrt{-1} \sin. q\, x)$$
$$Q = e^{px} (-\sin. q\, x + \sqrt{-1} \cos. q\, x)$$

and from these the two solutions will be

$$\begin{cases} P = e^{px} \cos. q\, x & Q = -e^{px} \sin. q\, x \\ P = e^{px} \sin. q\, x & Q = e^{px} \cos. q\, x \end{cases}$$

The second value of α in Equation (9) gives

$$P = e^{-px} (\cos. q\, x + \sqrt{-1} \sin. q\, x)$$
$$Q = e^{-px} (\sin. q\, x - \sqrt{-1} \cos. q\, x)$$

and the two solutions

$$\begin{cases} P = e^{-px} \cos. q\, x & Q = e^{-px} \sin. q\, x \\ P = e^{-px} \sin. q\, x & Q = -e^{-px} \cos. q\, x \end{cases}$$

Adding these four solutions, multiplied by constants, the general solution of the factors, P and Q, of Equation (1) is obtained.

$$\left.\begin{aligned} P &= e^{px} (C \cos. q\, x + D \sin. q\, x) + e^{-px} (C' \cos. q\, x + D' \sin. q\, x) \\ Q &= e^{px} (-C \sin. q\, x + D \cos. q\, x) + e^{-px} (C' \sin. q\, x - D' \cos. q\, x) \end{aligned}\right\} \ldots (10)$$

The value of α, obtained by changing q to $-q$, would give the same result.

Substituting these values of P and Q in Equation (1), the expression for the horizontal displacement will read

$$\begin{aligned} \xi = e^{px} &[C \cos. (\sigma\, t + q\, x) + D \sin. (\sigma\, t + q\, x)] \\ &+ e^{-px} [C' \cos. (\sigma\, t - q\, x) - D' \sin. (\sigma\, t - q\, x)] \ldots (11) \end{aligned}$$

To every expression found for ξ another corresponds for the height, h, and, by Equation (IV),

$$\begin{aligned} h = &-\gamma\, e^{px} [(C p + D q) \cos. (\sigma t + q x) + (D p - C q) \sin. (\sigma t + q x)] \\ &+ \gamma\, e^{-px} [(C' p - D' q) \cos. (\sigma t - q x) + (D' p + C' q) \sin. (\sigma t - q x)] \ldots (12) \end{aligned}$$

The numerical values of the constants depend on the particular conditions of the problem.

5.—Determination of the Constants in a Canal Without Proper Tide, Communicating at One End with a Tideless Lake and at the

Other End with a Tidal Sea.—Instead of the constants, C, D, C', D', of Equations (*11*) and (*12*), let there be introduced four new constants, A, B, A', B', being related to the former as follows:

$$C\,p + D\,q = \frac{A}{\gamma} \qquad C'\,p - D'\,q = \frac{A'}{\gamma}$$

$$-C\,q + D\,p = \frac{B}{\gamma} \qquad C'\,q + D'\,p = \frac{B'}{\gamma}$$

and, therefore,

$$\left.\begin{array}{ll} C = \dfrac{p\,A - q\,B}{\gamma\,(p^2+q^2)} & D = \dfrac{q\,A + p\,B}{\gamma\,(p^2+q^2)} \\ C' = \dfrac{p\,A' + q\,B'}{\gamma\,(p^2+q^2)} & D' = \dfrac{-q\,A' + p\,B'}{\gamma\,(p^2+q^2)} \end{array}\right\} \ldots\ldots (13)$$

Consequently, Equations (*11*) and (*12*) will become

$$\xi = \frac{1}{\gamma\,(p^2+q^2)}\left\{\begin{array}{l} e^{px}\,[(p\,A - q\,B)\cos.(\sigma\,t + q\,x) + (q\,A + p\,B) \\ \sin.\,(\sigma\,t + q\,x)] + e^{-px}\,[(p\,A' + q\,B') \\ \cos.\,(\sigma\,t - q\,x) + (q\,A' - p\,B') \\ \sin.\,(\sigma\,t - q\,x)] \end{array}\right\} \ldots (14)$$

$$h = e^{px}\,[-A\cos.(\sigma\,t + q\,x) - B\sin.(\sigma\,t + q\,x)]$$
$$+ e^{-px}\,[A'\cos.(\sigma\,t - q\,x) - B'\sin.(\sigma\,t - q\,x)] \ldots\ldots (15)$$

Let the origin of the abscissas, x, be at the lake end of the canal. Then we must have

$$\text{For } x = 0,\ h = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (o)$$

$$\text{For } x = L,\ h = h_0 \sin.\,\sigma\,t \ldots\ldots\ldots\ldots\ldots\ldots (r)$$

where $L =$ the length of the canal, h_0 the given half amplitude of the sinoidal tide, and $\sigma = \frac{2\,\pi}{T}$, if T is the interval between consecutive high tides. The origin of times is taken at $\frac{T}{4}$ preceding high water at the sea end.

To determine the constants, it is known from the condition in Equation (o) that

$$A' - A = 0 \text{ and } B' + B = 0,$$

and from the condition in Equation (r) that

$$e^{pL}\,(-A\cos.\,q\,L - B\sin.\,q\,L) + e^{-pL}\,(A'\cos.\,q\,L$$
$$+ B'\sin.\,q\,L) = 0.$$

$$e^{pL}\,(A\sin.\,q\,L - B\cos.\,q\,L) + e^{-pL}\,(A'\sin.\,q\,L$$
$$- B'\cos.\,q\,L) = h_0.$$

If there be put, for convenience,

$$\varDelta = 2\left(\frac{e^{2pL} + e^{-2pL}}{2} - \cos. 2\,q\,L\right) \ldots\ldots\ldots\ldots (16)$$

then

$$\left.\begin{aligned} A = A' &= \frac{h_0}{\varDelta}\,(e^{pL} + e^{-pL}) \sin. q\,L \\ B = -B' &= -\frac{h_0}{\varDelta}\,(e^{pL} - e^{-pL}) \cos. q\,L \end{aligned}\right\} \ldots\ldots (17)$$

and, substituting these values in Equations (14) and (15),

$$\xi = \frac{h_0}{\gamma\,\varDelta\,(p^2+q^2)}\left\{\begin{aligned} &p\left\{\begin{aligned} &-e^{p(L+x)} \sin. [\sigma\,t - q\,(L-x)] \\ &+e^{-p(L-x)} \sin. [\sigma\,t + q\,(L+x)] \\ &-e^{p(L+x)} \sin. [\sigma\,t - q\,(L+x)] \\ &+e^{-p(L-x)} \sin. [\sigma\,t + q\,(L-x)] \end{aligned}\right\} \\ +\,&q\left\{\begin{aligned} &e^{p(L+x)} \cos. [\sigma\,t - q\,(L-x)] \\ &-e^{-p(L-x)} \cos. [\sigma\,t + q\,(L+x)] \\ &+e^{p(L+x)} \cos. [\sigma\,t - q\,(L+x)] \\ &-e^{-p(L-x)} \cos. [\sigma\,t + q\,(L-x)] \end{aligned}\right\} \end{aligned}\right\} \quad (18)$$

$$h = \frac{h_0}{\varDelta}\left\{\begin{aligned} &e^{p(L+x)} \sin. [\sigma\,t - q\,(L-x)] \\ &-e^{-p(L-x)} \sin. [\sigma\,t + q\,(L+x)] \\ &-e^{p(L-x)} \sin. [\sigma\,t - q\,(L+x)] \\ &+e^{-p(L+x)} \sin. [\sigma\,t + q\,(L-x)] \end{aligned}\right\} \ldots\ldots\ldots (19)$$

The current will have the expression

$$\frac{\delta\,\xi}{\delta\,t} = \frac{h_0}{\gamma\,\varDelta\,(p^2+q^2)}\left\{\begin{aligned} &p\left\{\begin{aligned} &-e^{p(L+x)} \cos. [\sigma\,t - q\,(L-x)] \\ &+e^{-p(L-x)} \cos. [\sigma\,t + q\,(L+x)] \\ &-e^{p(L+x)} \cos. [\sigma\,t - q\,(L+x)] \\ &+e^{-p(L-x)} \cos. [\sigma\,t + q\,(L-x)] \end{aligned}\right\} \\ +\,&q\left\{\begin{aligned} &-e^{p(L+x)} \sin. [\sigma\,t - q\,(L-x)] \\ &+e^{-p(L-x)} \sin. [\sigma\,t + q\,(L+x)] \\ &-e^{p(L+x)} \sin. [\sigma\,t - q\,(L+x)] \\ &+e^{-p(L-x)} \sin. [\sigma\,t + q\,(L-x)] \end{aligned}\right\} \end{aligned}\right\} .(20)$$

These equations show that the motion in the canal consists of the superposition of four waves, two of which are propagated from the sea toward the lake, having a speed of $-\frac{\sigma}{q}$, and two in the opposite

direction, with a speed of $+\frac{\sigma}{q}$ The absolute value of these speeds, according to Equation (*8*) is

$$\frac{\sigma}{q} = \sqrt{\frac{2\,g\,\gamma}{1+\sqrt{1+\frac{f^2}{\sigma^2}}}} \quad\ldots\ldots\ldots\ldots\ldots\ldots(21)$$

which, if friction is not considered, reduces to the well-known value of $\frac{\sigma}{q} = \sqrt{g\,\gamma}$.

The four waves, which all have the same period, $\frac{2\,\pi}{\sigma} = T$, as the tidal sea, can be united into one, the height of which, h, is given by the equation:

$$h = P \cos.\, \sigma\, t + Q \sin.\, \sigma\, t \quad\ldots\ldots\ldots\ldots\ldots\ldots(22)$$

or, putting $\frac{P}{Q} = \tan.\, Z$, there can also be written

$$h = \sqrt{P^2 + Q^2}\, \sin.\, (\sigma\, t + Z) \quad\ldots\ldots\ldots\ldots\ldots\ldots(23)$$

In this equation

$$\left.\begin{array}{l} P = \frac{h_0}{\Delta}\left\{\begin{array}{l} -\,e^{p\,(L+x)} \sin.\, q\,(L-x) \\ -\,e^{-p\,(L-x)} \sin.\, q\,(L+x) \\ +\,e^{p\,(L-x)} \sin.\, q\,(L+x) \\ +\,e^{-p\,(L+x)} \sin.\, q\,(L-x) \end{array}\right\} \\ Q = \frac{h_0}{\Delta}\left\{\begin{array}{l} e^{p\,(L+x)} \cos.\, q\,(L-x) \\ -\,e^{-p\,(L-x)} \cos.\, q\,(L+x) \\ -\,e^{p\,(L-x)} \cos.\, q\,(L+x) \\ +\,e^{-p\,(L+x)} \cos.\, q\,(L-x) \end{array}\right\} \end{array}\right\} \quad\ldots\ldots\ldots(24)$$

If we substitute the hyperbolic functions,

$\sin._h\, x = \frac{e^{x} - e^{-x}}{2}$, $\cos._h\, x = \frac{e^{x} + e^{-x}}{2}$, $\tan._h\, x = \frac{\sin._h\, x}{\cos._h\, x}$, then

$$\left.\begin{array}{l} P = \frac{2\,h_0}{\Delta}\left\{\begin{array}{l} [\sin._h\, p\,(L-x)\, \sin.\, q\,(L+x) \\ \quad -\sin._h\, p\,(L+x)\, \sin.\, q\,(L-x)] \end{array}\right\} \\ Q = \frac{2\,h_0}{\Delta}\left\{\begin{array}{l} [-\cos._h\, p\,(L-x)\, \cos.\, q\,(L+x) \\ \quad +\cos._h\, p\,(L+x)\, \cos.\, q\,(L-x)] \end{array}\right\} \end{array}\right\} \quad\ldots\ldots(25)$$

The condition for the occurrence of maximum elevation at the section, x, is evidently that $\sin.\,(\sigma\, t + Z) = 1$; for this case

$$h_{max.} = h_0 \sqrt{\frac{\cos._h\, 2\,p\,x - \cos.\, 2\,q\,x}{\cos._h\, 2\,p\,L - \cos.\, 2\,q\,L}} \quad\ldots\ldots\ldots\ldots(26)$$

The time of this maximum will be found from the condition, $\sigma t + Z = \frac{\pi}{2}$

$$t = \frac{\pi}{2\,\sigma} - \frac{Z}{\sigma} = \frac{T}{4} - \frac{Z}{2\,\pi}\,T \ldots\ldots\ldots\ldots\ldots (27)$$

or, in other words, high water at the section, x, will occur $-\frac{Z}{2\,\pi}\,T$ later than at the sea.

The value of $Z = \frac{P}{Q}$, with respect to Equations (*16*), (*23*), and (*25*), which also furnished Equation (*26*), will be found from

$$\tan.\, Z = \frac{-\tan.\, q\, L \tan._h p\, x + \tan._h p\, L \tan.\, q\, x}{\tan._h p\, L \tan._h p\, x + \tan.\, q\, L \tan.\, q\, x} \ldots\ldots (28)$$

The expression of the current velocity also can be written in the form

$$\frac{\delta\, \xi}{\delta\, t} = M \cos.\, \sigma\, t + N \sin.\, \sigma\, t = \sqrt{M^2 + N^2} \sin.\, (\sigma\, t + Y) \ldots (29)$$

where $\tan.\, Y = \frac{M}{N}$, and M and N are given as follows:

$$\left.\begin{aligned} M &= \frac{2\, h_0\, \sigma}{\gamma\, \Delta\, (p^2 + q^2)} \left\{ \begin{aligned} &p\, [\sin._h p\, (L + x) \cos.\, q\, (L - x) \\ &+ \sin._h p\, (L - x) \cos.\, q\, (L + x)] \\ &+ q\, [\,-\cos._h p\, (L + x) \sin.\, q\, (L - x) \\ &- \cos._h p\, (L - x) \sin.\, q\, (L + x)] \end{aligned} \right\} \\ N &= \frac{2\, h_0\, \sigma}{\gamma\, \Delta\, (p^2 + q^2)} \left\{ \begin{aligned} &p\, [\cos._h p\, (L + x) \sin.\, q\, (L - x) \\ &+ \cos._h p\, (L - x) \sin.\, q\, (L + x)] \\ &+ q\, [\sin._h p\, (L + x) \cos.\, q\, (L - x) \\ &+ \sin._h p\, (L - x) \cos.\, q\, (L + x)] \end{aligned} \right\} \end{aligned}\right\} \ldots (30)$$

The value of the maximum current will be $= \sqrt{M^2 + N^2}$, and will occur when $\sin.\, (\sigma\, t + Y) = 1$, or $\sigma\, t + Y = \frac{\pi}{2}$,

and
$$t = \frac{T}{4} - \frac{Y}{2\,\pi}\,T \ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

6.—General Remarks.—The theory evolved in the preceding paragraphs and the equations presented give a comparatively easy and, for all practical purposes, correct solution of the problem of tidal phenomena in canals, if the basic assumptions are approximated in practice.

The tidal variations at the end of the canal can generally be represented very closely by an equation having the following form:

$$h = h_0 \sin.\, \sigma\, t + h_0' \sin.\, \sigma'\, t' + \ldots\ldots\ldots\ldots\ldots (32)$$

The equations in this paper furnish the solution for each member of the right-hand side of Equation (*32*), and these results, added together by the principle of superposition of waves, will solve the problem. If both ends of the canal are subjected to tidal influences, they should be treated separately and the results added.

If there is a difference between the mean elevations of the two seas, the influence of a uniform motion resulting from this circumstance should be added to the results obtained for the oscillating motion. If a_0 = the elevation of mean sea level at one end of the canal, and b_0 = that at the other end, then at any point of the canal, characterized by the abscissa, x, the elevation of the water, $a = a_0 - \frac{a_0 - b_0}{L} x$, and the value of the uniform velocity can be expressed approximately by the equation, $v = \frac{g\, I}{f}$, where $I = \frac{a_0 - b_0}{L}$, or by any equation derived for uniform flow in canals. If the canal is not very long, and the difference in mean sea levels is considerable, the equations for non-uniform but permanent flow should be applied. Later, in this paper, these equations will be dealt with in some detail.

In deriving the equations the assumption was made that the friction, F, is proportional to the first power of the velocity of the flow. Strictly speaking, therefore, they can be applied with precision only on canals of great length subjected to small differences in head, that is, in which the velocities are small enough to make the condition prevail. The integration of the Differential Equation (*I*) in cases where the power, n, of the velocity to which the frictional resistance is proportional differs from unity still awaits analytical solution.

To the writer's knowledge, no attempt has been made as yet to compensate for this deficiency and to amend the equations in such a way that they could be used for canals of the class of the Cape Cod Canal, in which the velocity is such that F is proportional to about the square of the velocity. This is probably due to the fact that no actual case has arisen, up to the present time, where any proposed theory could be substantiated by the results of careful measurements, and that for all tidal canals in existence, approximate calculations by equations based on Bernoulli's theorem give velocities sufficiently close for practical purposes.

In an entirely different field of hydrodynamics, however, it was imperative to evaluate the influence of the frictional resistances, when

proportional to the square of the velocity, on the oscillating motion of fluids. With the advent of long pipe lines supplying hydraulic power plants, means had to be found for the proper regulation of the wheels and for the protection of the conduits. The surge-tank regulator was devised for this purpose, and its correct design necessitated the prediction of the magnitude of the rise and fall of the water in it for the assumed operating conditions.

Within the last 8 or 10 years a number of books and papers have been published on this subject, mostly from the pens of American and Swiss engineers and scientists.* It can be stated that the problem of the surge tank is, to an extent, the inverse of the problem of the current in a tidal canal. In the former, from known ultimate velocity changes in the conduit, the oscillating motion induced within the reservoir at the end of the pipe is sought; and in the latter, the velocity changes in the canal due to the known harmonic changes of the head at the end are investigated.

The differential equation characterizing the motions of the water in the surge tank is, as could be inferred from the foregoing considerations, similar to Equation (*I*) in this paper, and the investigators of the surges were confronted by the same difficulty, namely, the impossibility of the analytical integration of the term, $\int v^2\, ds$. The evaluation of this integral, however, was absolutely necessary, in view of the more and more frequent application of the surge-tank regulator, and the question was attacked in both a theoretical and practical way. I. P. Church, Assoc. Am. Soc. C. E., in discussing Mr. Johnson's† and Mr. Warren's‡ papers, has shown that this expression can be integrated graphically and results derived with a great degree of accuracy. Other writers, as Dr. Prasil and Robert Dubs,§ have shown that the tedious graphical method can be avoided and excellent results obtained by using an average value of

* "Wasserschloss Probleme," by Professor F. Prasil, *Schweizerische Bauzeitung*, Vol. LII, No. 21 and following.

"Allgemeine Theorie über die veränderliche Bewegung des Wassers in Leitungen," by Robert Dubs and V. Bataillard, Berlin, 1909.

"The Surge Tank in Water Power Plants," by R. D. Johnson, *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, 1908.

"The Differential Surge Tank," by R. D. Johnson, *Transactions*, Am. Soc. C. E., Vol. LXXVIII, 1915.

"Penstock and Surge-Tank Problems," by Minton M. Warren, *Transactions*, Am. Soc. C. E., Vol. LXXIX, 1915.

† *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, p. 488 and following.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 273 and following.

§ "Allgemeine Theorie," etc., II Teil: Stollen und Wasserschloss, pp. 219–221.

the velocity, such that the frictional force, as a linear function of the same, will absorb the same total energy in the interval during which the velocity of the conduit changes from zero to maximum, as the total energy absorbed by the frictional force in the same interval, regarding F as proportional to the square of the velocity at any instant. This simplified method is endorsed by Mr. Johnson,* and a great number of experiments have proved the reliability of its use.

On the strength of the foregoing argument, the writer proposes the following modification of Equations (c) and (III) and those deduced therefrom.

In the case of variable motion, where the velocity ranges from zero to a considerable value, as in the Cape Cod Canal, for the value of f_0, as given in textbooks on hydraulics for different consistency of the wetted perimeter, a different value, f_0', should be substituted, so that the equation

$$F_{average} = f_0' \, v_{mean} \quad \ldots\ldots\ldots\ldots\ldots\ldots (c_1)$$

will be satisfied.

For streams of the description of the Cape Cod Canal, $n = 2$ in the equation, $F = f_0 \, v^n$. Now, $F = f_0 v^2$ can be represented by the ordinates of a parabola, the average ordinate of which is 75% of the maximum. Due to the fact disclosed by more recent investigations that f_0 itself is not a constant quantity but varies slightly inversely with the velocity, it is proposed to use the value, $F_{average} = 0.7 \, F_{max.}$, and therefore $f_0' = \frac{0.7 \, F_{max.}}{0.5 \, v_{max.}}$, $0.5 \, v_{max.}$ being the mean velocity. The value of $v_{max.}$, to be used in such cases, can be established by any of the approximate methods to be described later. For maximum velocities less than 1.5 ft., the original assumption can be considered as correct, and the use of f_0 as given for 1 ft. velocity is recommended, it being a well-known fact that at such low velocities the friction is actually proportional to the velocity.

If the maximum velocity computed by the submitted equations would considerably differ from the velocity given by the approximate methods, the value of f_0' should be corrected for the former, and the whole calculation refigured with this new value of f_0'. This process may be repeated until a satisfactory agreement between the figured value of $v_{max.}$ and that of f_0' is reached.

* *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, p. 455.

Mr. Lévy, in applying the equations to the Suez Canal, used a coefficient, $\frac{f_0}{\eta} = 0.0005$, and found that the computed are far below the actual velocities, and therefore suggested the use of equations based on the extension of Equation (*III*) to a second approximation. Judging from the close check of the computed and observed velocities of the Cape Cod Canal, the equations herein presented give reliable results for all practical purposes, if the value of f_0' is taken in accordance with the previous reasoning. In the case of the Suez Canal, for the conditions considered by Mr. Lévy, the approximate methods furnish a $v_{max.}$ of about 2.5 ft. For the consistency of the Suez Canal bed, f_0 can be taken as 0.004. Then

$$f_0' = \frac{0.7 \times 0.004 \times 2.5^2}{1.25} = 0.014, \text{ and } \frac{f_0'}{\eta} = 0.00022,$$

which value gives velocities very close to the observed results in the Suez Canal.

Dr. Rollin A. Harris* gives the following values of f_0 for different consistencies of the wetted perimeter:

Fine sand	0.00405	These values are called the Eytelwein frictional values.
Coarse sand	0.00488	
Beds of streams........	0.00756	

7.—*Application of the Theory to the Cape Cod Canal.*—In order to test the validity of the equations supposed to furnish the values for the elevation of the water and velocity at any section and time, the equations must be applied to a stretch of uniform or nearly uniform cross-section. Such a stretch, according to the general plan, Fig. 2, extends from about Station 74 to Station 382. The nearest points, where observations were taken on August 26th, 1915, are Station 80 and Station 375, which we will consider as the ends of the canal having a uniform bottom width of 100 ft., so that we will make the length of the canal $L = 29\,500$ ft.

The mareographs of these stations, taken from the observations, are shown on Fig. 32, together with the sine curves having the same periods and amplitudes, and which are considered in the computations as being the tidal variations affecting the canal. It will be noted that the sine curve for Station 80, drawn with mean sea level at Elevation 100, is very close to the actual curve for the whole range

* "Manual of Tides," Part V., p. 249.

of the observation; but, at Station 375, though Elevation 100 divides the period into two almost equal portions, the second part of the cycle has a very much smaller amplitude than the first. This is typical for the westerly end, for reasons previously explained.

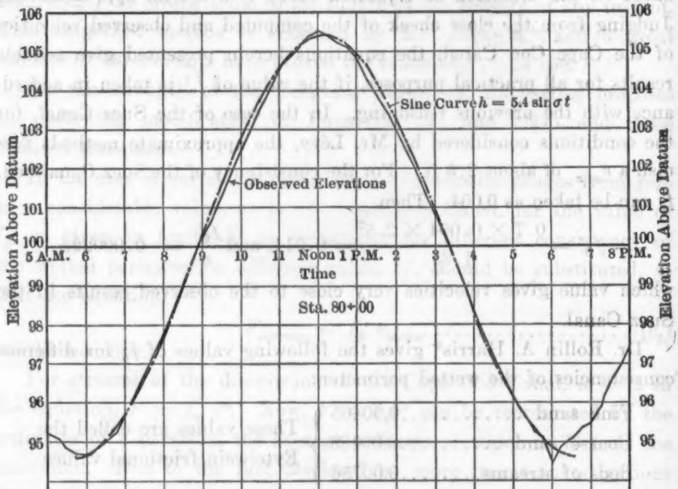

FIG. 32.

The bottom of the canal is at Elevation 70.35 at Station 80 and at Elevation 72.35 at Station 375, having a practically uniform slope. The reduced depths of the two end sections differ only by about 3% from the reduced depth based on a mean section having the bottom at Elevation 71.35, and therefore it may be considered justifiable to use this mean section (see Fig. 33*) as applying to the whole canal, instead of going into the exceedingly more complicated question of

* Fig. 33 is not drawn to scale.

solving the tidal motions in a canal of variable depth. It is evident that, with the degree of approximation permitted throughout in the derivation of the equations, this substitution is permissible.

It is also assumed that the direct effect of the moon and sun on the water in the canal is negligible.

The bed of the canal consists of coarse gravel and boulders, and the broken-stone slope protection tends to make the wetted perimeter still rougher. Considerable energy is expended by the water in scouring the bottom and transporting the material. In other words, the canal comes under the same group as ordinary streams, and the selection of the corresponding coefficient $f_0 = 0.00756$, seems to be justified. The same coefficient was adopted by the officers of the United States Coast and Geodetic Survey when asked to predetermine the velocities in the Cape Cod Canal. In approximate calculations, referred to later, it has been found that, for conditions prevailing at the time of the observations on August 26th, 1915, the maximum velocity in the canal is not expected to exceed 5 ft. per sec. Therefore,

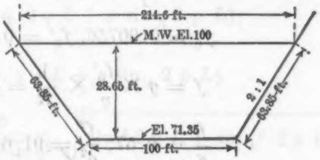

FIG. 33.

$$F_{max.} = f_0\, v^2 = 0.00756 \times 5^2 = 0.189 \text{ lb. per sq. ft.}$$

$$f_0' = \frac{0.7\, F_{max.}}{0.5\, v_{max.}} = \frac{0.7 \times 0.189}{2.5} = 0.528 \text{ lb. per cu. ft.}$$

In the following will be presented the numerical calculations for establishing the velocities at Stations 80, 227 + 50, and 375, and the water elevations at Station 227 + 50, on the basis of the tidal observations of August 26th, 1915. These calculations are divided into four parts. Under (*A*) the determinations of the constant quantities entering into the computations are given; under (*B*) and (*C*) the influences of the tides at the two ends are treated separately; under (*D*) the superposition of the water elevations and current velocities, as found under (*B*) and (*C*), is shown graphically.

(*A*).—*Determination of Constant Quantities.*—

g = acceleration of gravity.................... 32.2 ft.

π = weight of 1 cu. ft. of sea water............. 64.0 lb.

Z = wetted perimeter of undisturbed canal........ 277.7 ft.

Ω = area of undisturbed cross-section............ 4 510.0 sq. ft.

R = hydraulic radius..19.8 ft.

γ = reduced depth..21.0 ft.

L = length of canal..29 500.0 ft.

T = interval between consecutive high tides:

Station 80..............................12 hr. 54 min.

Station 375..............................12 hr. 24 min.

$$\sigma = \frac{2\pi}{T} = 0.00014 \text{ radian per second};$$

$$f_0 = 0.00756,\ f_0' = 0.0528,\ \frac{f_0'}{\eta} = 0.000825;$$

$$f = g \times \frac{f_0'}{\eta} \times \frac{1}{R} = 0.00134;$$

$$\frac{f}{\sigma} = 9.57,\ \frac{f^2}{\sigma^2} = 91.6;$$

$$p = \frac{\sigma}{\sqrt{2\,g\,\gamma}} \sqrt{-1 + \sqrt{1 + 91.6}} = 0.0000111;$$

$$q = \frac{\sigma}{\sqrt{2\,g\,\gamma}} \sqrt{1 + \sqrt{1 + 91.6}} = 0.0000124;$$

$$\sqrt{p^2 + q^2} = 0.0000167,\ \Delta = 2\,(\cos._h\ 2\,p\,L - \cos.\ 2\,q\,L)$$

$$= 0.96,\ \sqrt{2\,g\,\gamma} = 36.75$$

TABLE 15.

		arc.	sin.	cos.	tan.	sin.$_h$	cos.$_h$	tan.$_h$
For $x = L$ = 29 500 ft.	$p\,L = 0.328$	18° 50′	0.323	0.946	0.341	0.334	1.054	0.317
	$q\,L = 0.366$	21° 00′	0.358	0.934	0.384	0.375	1.068	0.351
	$2\,p\,L = 0.656$	37° 40′	0.611	0.792	0.772	0.704	1.223	0.576
	$2\,q\,L = 0.732$	42° 00′	0.669	0.743	0.900	0.800	1.280	0.624
For x and $(L - x)$ = 14 750 ft.	$p\,x = 0.164$	9° 25′	0.164	0.986	0.166	0.165	1.013	0.163
	$q\,x = 0.183$	10° 30′	0.182	0.983	0.185	0.184	1.017	0.181
	$2\,p\,x = 0.328$	18° 50′	0.323	0.946	0.341	0.334	1.054	0.317
	$2\,q\,x = 0.366$	21° 00′	0.358	0.934	0.384	0.375	1.068	0.351
For $(L + x)$ = 44 250 ft.	$p\,(L + x) = 0.492$	28° 15′	0.473	0.881	0.537	0.512	1.124	0.456
	$q\,(L + x) = 0.549$	31° 30′	0.523	0.853	0.613	0.577	1.155	0.500
	$2\,p\,(L + x) = 0.984$	56° 30′	0.834	0.552	1.511	1.151	1.524	0.755
	$2\,q\,(L + x) = 1.098$	63° 00′	0.891	0.454	1.963	1.332	1.667	0.800

(B).—Height of Tide, Current Velocities, and Retardations in the Canal, Due to Tidal Variations at Station 80.—

h_0 = half amplitude = 5.4 ft.

(*a*) At Station 80..............$x = L = 29\,500$ ft.

The elevation of the water at any time must be the same as the mareograph at Station 80, and is given by $h = h_0 \sin. \sigma\, t$, where the zero time is $\frac{T}{4}$ before high water.

The current velocity:

$v = \sqrt{M^2 + N^2} \sin. (\sigma\, t + Y)$ according to Equation (29), where from Equation (30),

$$M = \frac{2\, h_0\, \sigma}{\gamma\, \Delta\, (p^2 + q^2)} (-\, q \sin. 2\, q\, L + p \sin._h 2\, p\, L),$$

$$N = \frac{2\, h_0\, \sigma}{\gamma\, \Delta\, (p^2 + q^2)} (p \sin. 2\, q\, L + q \sin._h 2\, p\, L)$$

$$v_{max.} = \sqrt{M^2 + N^2} = \frac{2\, h_0\, \sigma}{\gamma\, \Delta \sqrt{p^2 + q^2}} \sqrt{\sin.^2 2\, q\, L + \sin._h{}^2\, 2\, p\, L}$$

$$= \frac{2 \times 5.4 \times 0.00014}{21 \times 0.96 \times 0.0000167} \sqrt{0.669^2 + 0.704^2}$$

$$= 4.35 \text{ ft. per sec.}$$

$$\tan. Y = \frac{M}{N} = \frac{-\, q \sin. 2\, q\, L + p \sin._h 2\, p\, L}{p \sin. 2\, q\, L + q \sin._h 2\, p\, L}$$

$$= \frac{-\, 0.0000124 \times 0.669 + 0.0000111 \times 0.704}{0.0000111 \times 0.669 + 0.0000124 \times 0.704} = -\, 0.0303$$

or $\quad Y = -\, 1° \, 45'$

$v_{max.}$ will occur, when $t = \frac{T}{4} - \frac{Y}{\sigma}$, and the retardation equals

$$\frac{1°45'}{360°} \times 46\,440 = 226 \text{ sec., or about 4 min.}$$

The maximum velocity, therefore, is retarded by 4 Min. after high water.

(*b*) At Station 227+50. . . . .$x = 14\,750$ ft.

The maximum elevation of the water is given by Equation (*26*)

$$h_{max.} = h_0 \sqrt{\frac{\cos._h 2\, p\, x - \cos. 2\, q\, x}{\cos._h 2\, p\, L - \cos. 2\, q\, L}} = 5.4 \times 0.5 = 2.70 \text{ ft.}$$

The time of this maximum will be found by Equation (*28*),

$$\tan. Z = \frac{-\tan. q\, L \tan._h p\, x + \tan._h p\, L \tan. q\, x}{\tan._h p\, L \tan._h p\, x + \tan. q\, L \tan. q\, x}$$

$$= \frac{-\, 0.384 \times 0.163 + 0.317 \times 0.185}{0.317 \times 0.163 + 0.384 \times 0.185} = -\, 0.0322, \text{ and } Z = -\, 1°50'.$$

The retardation, therefore, $= \frac{1° \, 50'}{360°} \times 46\,440$, or about 4 min.

The current velocity:

Again, $v = \sqrt{M^2 + N^2} \sin. (\sigma t + Y)$, where, from Equation (30), denoting

$\sin._h p (L + x) \cos. q (L - x)$ by $a = 0.512 \times 0.983 = 0.503$
$\sin._h p (L - x) \cos. q (L + x)$ by $b = 0.165 \times 0.853 = 0.141$
$\cos._h p (L + x) \sin. q (L - x)$ by $c = 1.124 \times 0.182 = 0.205$
$\cos._h p (L - x) \sin. q (L + x)$ by $d = 1.013 \times 0.523 = 0.530$

we can write

$$M = \frac{2 h_0 \sigma}{\gamma \varDelta (p^2 + q^2)} [p (a + b) - q (c + d)],$$

$$N = \frac{2 h_0 \sigma}{\gamma \varDelta (p^2 + q^2)} [p (c + d) + q (a + b)]$$

$$v_{max.} = \sqrt{M^2 + N^2} = \frac{2 h_0 \sigma}{\gamma \varDelta \sqrt{p^2 + q^2}} \sqrt{(a + b)^2 + (c + d)^2}$$

$$= \frac{2 \times 5.4 \times 0.00014}{21 \times 0.96 \times 0.0000167} \sqrt{0.644^2 + 0.735^2} = 4.4 \text{ ft. per sec.}$$

$$\tan. Y = \frac{M}{N} = \frac{p (a + b) - q (c + d)}{p (c + d) + q (a + b)}$$

$$= \frac{0.0000111 \times 0.644 - 0.0000124 \times 0.735}{0.0000111 \times 0.735 + 0.0000124 \times 0.644} = -0.122,$$

and $$Y = -7°.$$

The retardation equals $\frac{7°}{360°} \times 46\,440 = 910$ sec., or about 15 min.

(*c*) At Station 375 . . . . $x = 0$.

The elevation of the water is given by $h = 0$.

The current velocity:

$$v_{max.} = \frac{4 h_0 \sigma}{\gamma \varDelta \sqrt{p^2 + q^2}} \sqrt{\cos.^2 q L \sin._h^2 p L + \sin.^2 q L \cos._h^2 p L}$$

$$= \frac{4 \times 5.4 \times 0.00014}{21 \times 0.96 \times 0.0000167} \sqrt{0.934^2 \times 0.334^2 + 0.358^2 \times 1.054^2}$$

$$= 4.31 \text{ ft. per sec.}$$

$$\tan. Y = \frac{p \cos. q L \sin._h p L - q \sin. q L \cos._h p L}{p \sin. q L \cos._h p L + q \cos. q L \sin._h p L}$$

$$= \frac{1.0000111 \times 0.934 \times 0.334 - 0.0000124 \times 0.358 \times 1.054}{0.0000111 \times 0.358 \times 1.054 + 0.0000124 \times 0.934 \times 0.334}$$

$$= -0.15, \text{ and } Y = 8° 30'.$$

The retardation, therefore, will be $\frac{8° 30'}{360°} \times 46\,440 = 1\,100$ sec., or about 18 min.

(*C*).—*Influence of the Tide at Station 375.*—

h_0 = half amplitude for first part of cycle = 3.05 ft.

h'_0 = " " " second " " " = 1.525 ft.

It is evident, by inspection of the equations, that the results found under (*B*) for retardations will be the same. For the height of water and the velocities, the corresponding figures should be multiplied by the ratio, $\frac{3.05}{5.4}$, for the first half, and by $\frac{1.525}{5.4}$ for the second half, of the cycle.

(*a*) At Station 80.......................$x = 0$.

The elevation of the water is given by $h = 0$.

The maximum velocity will be

$$4.31 \times \frac{3.05}{5.4} = 2.43 \text{ ft. per sec. for the first half}$$

and $4.31 \times \frac{1.525}{5.4} = 1.22$ ft. per sec. for the second half of the cycle.

The retardation of the maximum velocity will be 18 min.

(*b*) At Station 227 + 50.....................$x = 14\,750$ ft.

The maximum water elevation equals $2.70 \times \frac{3.05}{5.4} = 1.53$ ft. and $2.70 \times \frac{1.525}{5.4} = 0.77$ ft., respectively.

The retardation is 4 min.

The maximum velocity equals $4.4 \times \frac{3.05}{5.4} = 2.49$ ft. per sec. and $4.4 \times \frac{1.525}{5.4} = 1.25$ ft. per sec., respectively.

The retardation is 15 min.

(*c*) At Station 375.....................$x = L = 29\,500$ ft.

The elevation of the water is given by $h = h_0 \sin. \sigma t$.

The maximum velocity will be $4.35 \times \frac{3.05}{5.4} = 2.46$ ft. per sec. and $4.35 \times \frac{1.525}{5.4} = 1.23$ ft. per sec., respectively.

The retardation is 4 min.

(*D*).—*Combination of* (*B*) *and* (*C*).—Figs. 34, 35, and 36 show the velocities at Stations 80, 227 + 50, and 375, and Fig. 37 shows the water elevation at Station 227 + 50, all platted as ordinates to the

times, which are considered as the abscissas of the curves. The curves in full lines show the observed velocities reduced to mean velocities; the curves in dotted lines show the component velocities plotted as sine curves to the computed maximum values, which have been located according to the figured retardations; and the curves in lines of dashes and dots the resulting velocities, the components having been added graphically. The same method has been followed in preparing Fig. 37.

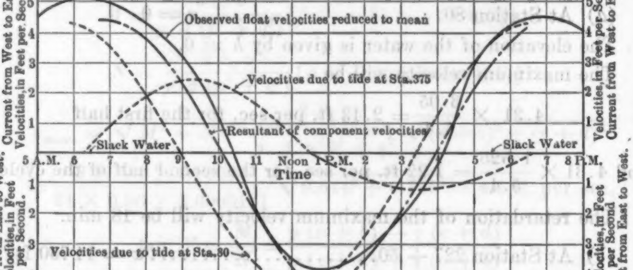

FIG. 34.

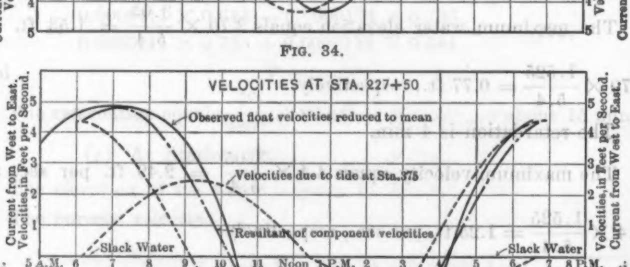

FIG. 35.

It should be noted that no observations were taken at Station 227 + 50, but they were made at Station 225, only 250 ft. distant. The latter have been taken for the sake of comparison.

The cross-section of the canal at Station 375 is actually very much larger than designed, the area below mean sea level being 5 900 sq. ft.

In platting the results for this station, therefore, the computed velocities have been multiplied by the ratio, $\frac{4\,510}{5\,900} = 0.765$.

8.—Results.—Inspection of Figs. 34, 35, 36, and 37 shows a remarkably close check between computed and observed conditions, in so far as the water elevations, maximum velocity of the current, and the time of maximum current and slack water are concerned. The greatest deviations for maximum velocities occur at Station 80, where they amount to more than 10 per cent. They are somewhat smaller at Station 375, and the results check almost exactly at Station 227 + 50.

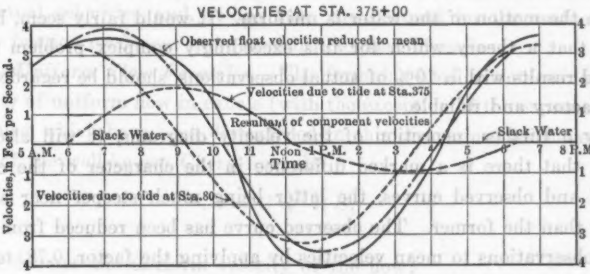

FIG. 36.

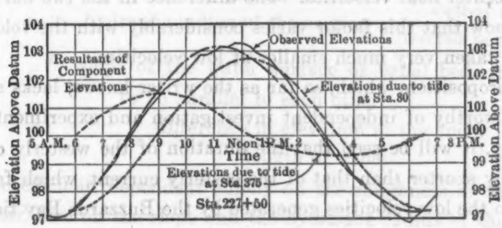

FIG. 37.

The discrepancy at the ends can be explained by the fact, as dealt with in detail in another part of this paper, that, at these stations, for outflow a smaller and for inflow a greater percentage of reduction should be applied to the float observations, in order to arrive at the true mean velocity of the cross-section. If the reduction factors of 68 and 85%, respectively, which are believed to be the true values for reducing the center float velocities for the terminus sections, had been applied to the observed velocities at Stations 80 and 375, the differences of the computed and the reduced velocities would be negligible at these sta-

tions also. If, furthermore, it is considered that the curves of observed velocities have been derived from float observations, which evidently are affected by accidental disturbances, such as the wind, the passing of larger boats, and local changes in the cross-section, it may be concluded that the computed results are well within 10% of the actual conditions.

It cannot be denied that the uncertainty attached to the selection of the friction coefficient cannot be entirely eliminated, and therefore one should not expect as close results as hydraulic engineers are accustomed to; for instance, in the case of masonry conduits or pipes in which the motion of the water is uniform. It would fairly seem, however, that a theory which for this exceedingly complex problem furnished results within 10% of actual observations, should be regarded as satisfactory and reliable.

By a further inspection of the velocity diagrams, it will also be noted that there is a marked difference in the character of the computed and observed curves, the latter being much steeper near slack water than the former. The observed curve has been reduced from the float observations to mean velocities by applying the factor, 0.78, to all observed center float velocities. The difference in the two curves would tend to show that this factor varies considerably with the velocity, and should be taken very much smaller at low velocities.

This proposition, which, so far as the writer knows, lacks authority, might be worthy of independent investigation and experimentation.

Finally, it will be seen that the duration of the westerly current is appreciably shorter than that of the easterly current, which fact is due directly to the low velocities generated by the Buzzards Bay tide during the second half of the cycle. It will also be noted that the computed time of maximum velocities and the computed time of slack water check within a few minutes with the actual occurrence of these extremes.

The equations submitted herewith are general, and their use is not restricted by any other consideration than a canal of reasonably uniform cross-section. They can also be used for predicting conditions in canals of varying cross-section, by dividing them into reaches, where the cross-sections can be considered as sensibly uniform, and by determining the constants, C, D, C', and D' in the General Equations (*11*) and (*12*) from the conditions that both ξ and h must be equal at the

limits of two consecutive reaches. This calculation, of course, will be very much more complicated than that shown in this paper, but it is the only way of getting reliable results in the case where the canal is long and the cross-section varies considerably.

Approximate Methods.

For a uniform canal connecting tidal bodies of water, approximate formulas to predict velocities may be used, with certain restrictions, giving results which for maximum conditions will not be very far from the actual velocities, and, at any rate, can be used for the evaluation of the friction coefficient to be used in the exact formulas.

9.—Uniform Flow Formulas.—The formulas used to determine the velocity of uniform flow in canals (with the exception of the exponential formulas, which have been disregarded in this discussion) all have the form of the well-known Chezy formula

$$v = C\sqrt{R\,S},$$

where

v denotes the uniform velocity of the flow;

R " " hydraulic radius of the section $= \dfrac{\text{wetted area}}{\text{wetted perimeter}}$;

S " " slope of the water surface or canal bed, supposed to be parallel to each other $= \dfrac{\text{difference in elevation at the ends}}{\text{length of canal}}$;

C is a coefficient, dependent chiefly on the roughness of the perimeter, also on the slope and the hydraulic radius of the cross-section.

The several formulas in use differ only in the different methods of establishing this coefficient.

In order to be able to apply these formulas to a canal in which tidal motion takes place, the fact must be disregarded that the bottom for most of the time has a slope different from that of the surface, and one must consider the surface slope, assumed to be a straight line, as being $= S$. An average value of R must be introduced into the formulas. These assumptions made, it is evident that the velocity becomes proportional to the square root of the difference in elevations

at the ends of the canal. In Fig. 38, therefore, are platted the mareographs at Stations 80 and 375 in such a way that the difference in levels can be read off for every time point. Being interested in the maximum value of the velocity, let the following formulas be applied to the maximum difference in levels, which occurred at 6.45 A. M., and amounted to 5.85 ft. The hydraulic radius of the canal at this time averages 18.6 ft., the stage of the water being very low.

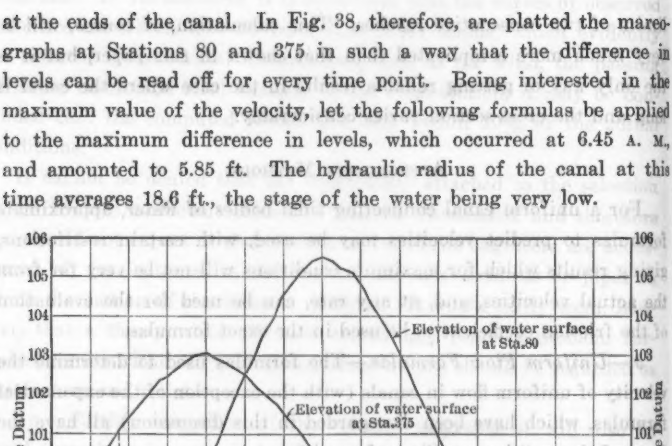

FIG. 38.

(*a*).—*Bazin's Formula.*—The great pioneer of hydraulic researchers, Bazin, has derived a formula, intended for the calculation of flow in open channels, in which it is assumed that the coefficient, C, does not vary sensibly with the slope. The formula (in foot units) reads

$$v = \frac{87}{0.552 + \dfrac{m}{\sqrt{R}}} \sqrt{RS} = C \sqrt{RS}$$

where the values of m, corresponding to the frictional constants given on page 121, are 0.85, 1.30, and 1.75, respectively.

With this last value of m, C becomes equal to 90.6, and

$$v = 90.6 \sqrt{18.6 \times \frac{5.85}{29\,500}} = 5.5 \text{ ft. per sec.}$$

(*b*).—*Eytelwein's Formula.**—This is written in a different form from the Chezy formula, expressing the velocity as a function of the head, that is, the difference in elevations at the ends of the canal.

$$v = \sqrt{\frac{2\,g}{1+\dfrac{K\,L}{R}}}\,\sqrt{h_1 - h_2}.$$

Of course, there is no difficulty in bringing this formula back to the form of the Chezy formula, in which case it would read

$$v = \sqrt{\frac{2\,g\,L}{R\left(1+\dfrac{K\,L}{R}\right)}}\,\sqrt{R\,S} = C\,\sqrt{R\,S}$$

where K is the frictional constant and g is the acceleration of gravity. With the value, $K = 0.00756$, and the other quantities substituted,

$$v = \sqrt{\frac{2 \times 32.2}{1+\dfrac{0.00756 \times 29\,500}{18.6}}}\,\sqrt{5.85} = 5.38 \text{ ft. per sec.}$$

(*c*).—*Kutter-Ganguillet Formula.*—This formula assumes the coefficient, C, to vary with the slope as well as with the roughness of the bed and the hydraulic radius. The expression for the velocity in this formula (in foot units) reads,

$$v = \frac{41.6 + \dfrac{1.811}{n} + \dfrac{0.00281}{S}}{1 + \left(41.6 + \dfrac{0.00281}{S}\right)\dfrac{n}{\sqrt{R}}}\,\sqrt{R\,S} = C\,\sqrt{R\,S}.$$

The values of n, corresponding to the frictional constants, are 0.02, 0.025, and 0.03, respectively.

With this last value of n, C becomes equal to 83, and

$$v = 83 \times \sqrt{18.6 \times \frac{5.85}{29\,500}} = 5.04 \text{ ft. per sec.}$$

This result was used for evaluating the coefficient, f_0', in the harmonic solution of the problem.

(*d*).—*Remarks.*—The application of the foregoing formulas to the Cape Cod Canal and the comparison of the results with the observations, show a reasonable check, in so far as maximum velocities are concerned. This was to be expected in the case of a comparatively

* This formula was used by the officers of the U. S. Coast and Geodetic Survey, when asked by the writer to predetermine the velocities in the Cape Cod Canal.

short canal, where the retardation of the wave due to friction amounts only to a few minutes, and therefore the maximum slope of the stream is sensibly equal to the maximum difference in water levels at the ends divided by the length of the canal. In the case of long canals, the formulas for uniform flow become unreliable, giving much lower results than the actual. In the case of short canals, and considerable differences in head, on the other hand, the formulas will become inaccurate because the basic assumptions that the slope of the water and that of the canal bed are nearly parallel will not be true. It can be stated, on the basis of trial calculations not reproduced here, that, for a canal similar to the Cape Cod Canal in cross-section and roughness of bed, the uniform-flow formulas will furnish too high or too low velocities, according as the length is such as to make $q L$ smaller or greater than 0.45.

VELOCITIES CUMPUTED BY DIFFERENT FORMULAS
COMPARED WITH OBSERVED VELOCITIES.

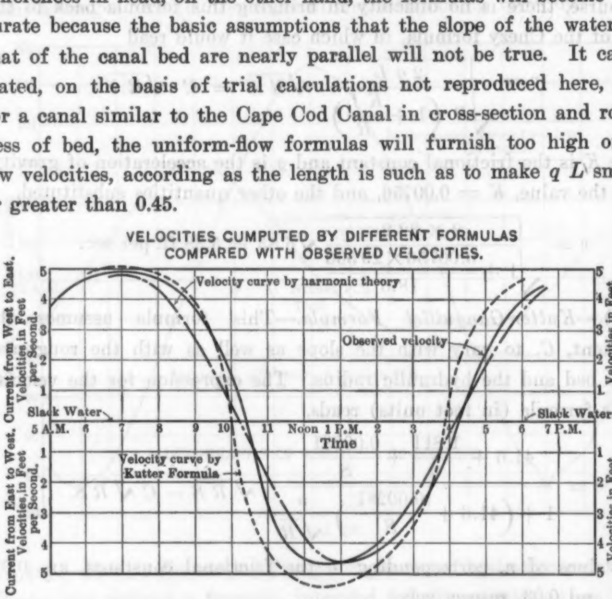

FIG. 39.

It can also be seen that the formulas for uniform flow can be applied only in cases where there is considerable difference in the amplitude of the tides at the two ends of the canal, or where there is a considerable difference in the phases of the tides. With equal tides and coincident phases, these formulas would furnish zero velocities for all sections, whereas, actually, there will be considerable motion at the ends of a canal several miles long, the velocities diminishing toward the middle of the canal. Fig. 39 shows the velocity in the Cape Cod Canal as computed by Kutter's formula for the whole range of observations on August 26th, 1915, as compared with the results by the modified Airy-Lévy formulas and the actual measurements.

(10).—Formula for Permanent Non-Uniform Flow.—Due to the fact that maximum velocity in the canal occurs near the time when the difference in elevations at the ends of the canal is a maximum, and due also to the fact that a change from this condition is at a comparatively slow rate, the canal for maximum velocity can be considered as one with very nearly horizontal bottom, connecting two seas of different elevations, discharging water from the higher to the lower.

The method of computation hereafter shown is given in "Handbuch der Ingenieurwissenschaften" by Professor Bubendey of Hamburg, but, to the writer's knowledge has never been published in English.

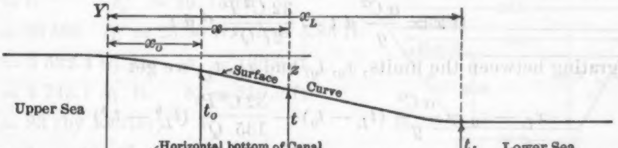

FIG. 40.

The differential equation of non-uniform flow in channels reads

$$\frac{d\,z}{d\,x} = -\frac{\alpha}{g}\frac{Q^2}{F^3}\frac{d\,F}{dx} + \frac{Q^2\,X}{F^3\,C^2},$$

where $Q =$ constant discharge of canal;
$F =$ area of cross-section;
$X =$ wetted perimeter of cross-section;
$g =$ acceleration of gravity;
$C =$ the constant in Chezy's formula, $v = C\sqrt{R\,S}$.
$\alpha =$ a constant, the value of which is 1.11 according to St. Venant.

The meaning of the other symbols is shown in Fig. 40.

There will first be derived the working formula for a parabolic cross-section and then be shown how a trapezoidal section can be transformed into an equivalent parabolic section. As an allowed approximation, assume that the width of the water surface, $b = X$, the wetted perimeter. Then

$$X = b = 2\sqrt{P\,t}$$

where $P =$ the parameter of the parabola and $t =$ the maximum

depth of the cross-section. From the well-known properties of the parabola,

$$F = \frac{4}{3}\sqrt{Pt}\;t, \text{ and } F^3 = \frac{64\,P^{\frac{3}{2}}\;t^{\frac{9}{2}}}{27}$$

$$\frac{d\,F}{d\,x} = 2\,P^{\frac{1}{2}}\;t^{\frac{1}{2}}\,\frac{d\,t}{d\,x},\;\frac{1}{F^3}\,\frac{d\,F}{d\,x} = \frac{27}{32\,P\,t^4}\,\frac{d\,t}{d\,x},\;\frac{X}{F^3} = \frac{27}{32\,P\,t^4}$$

As $dz = -\,dt$, according to the figure, substituting the other values in the differential equation, we get

$$\frac{d\,t}{d\,x} = \frac{27\,\alpha\,Q^2}{32\,g\,P\,t^4}\,\frac{d\,t}{d\,x} - \frac{27\,Q^2}{32\,C^2\,P\,t^4}$$

and from this, separating the variables,

$$d\,x = \frac{\alpha\,C^2}{g}\,d\,t - \frac{32\,C^2\,P}{27\,Q^2}\,t^4\,d\,t$$

Integrating between the limits, x_0, t_0, and x_L, t_L, we get

$$x_L - x_0 = \frac{\alpha\,C^2}{g}\,(t_L - t_0) - \frac{32\,C^2\,P}{135\;Q^2}\,(t_L{}^5 - t_0{}^5)$$

which equation is a complete solution of the problem. With given water elevations at the ends of the canal, and given dimensions of the cross-section, the discharge, Q, can be found. After Q is computed, the values of x corresponding to different values of t (between t_0 and t_L) can be found and the surface curve platted. With the help of the surface curve, the areas of the wetted cross-sections at any point can be measured, and consequently the velocities can be determined by dividing Q by the corresponding areas.

Let F_1, b_1, and F_2, b_2, be the area and surface width of the trapezoidal cross-sections at the outlet and inlet ends of the canal, respectively, then the parameter of the equivalent parabola is

$$P = \frac{1}{2}\left(\frac{b_1{}^3}{6\,F_1} + \frac{b_2{}^3}{6\,F_2}\right).$$

Let a_1' and a_1'' represent the distance of the vertex of the parabola from the lower sea level, at the outlet and inlet ends, respectively, and let h = the difference in levels, then

$$\frac{2}{3}\,a_1'\,b_1 = F_1 \text{ and } \frac{2}{3}\,(a_1'' + h)\,b_2 = F_2$$

and the average

$$a_1 = \frac{a_1' + a_1''}{2}.$$

If t_L' and t_0' are the actual depths of the trapezoidal cross-sections at the outlet and inlet, then

$$t_L = t'_L + (a_1 - t'_L) = a_1$$

and
$$t_0 = t_0' + (a_1 - t_L')$$

t_0 and t_L being the maximum depths of the substituted parabolic sections at the inlet and outlet, respectively.

Applying the formula for the conditions prevailing at the time of the maximum difference in levels on August 26th, 1915, the water elevations at Stations 80 and 375 are found to be 95.25 and 101.1, respectively, the current being easterly. These elevations being platted in Fig. 41,* the following data are obtained:

$x_0 = 0 \qquad t_0' = 29.75$ ft.

$x_L = 29\,500 \quad t_L' = 23.90$ ft. $\quad h = 5.85$ ft.

$F_1 = 3\,532.4$ sq. ft. $\quad b_1 = 195.6$ ft.

$F_2 = 4\,745.1$ sq. ft. $\quad b_2 = 219.0$ ft.

$C = 83$ (by Kutter).

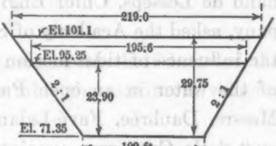

FIG. 41.

$$P = \frac{1}{2}\left(\frac{195.6^3}{6\times 3\,532.4} + \frac{219.0^3}{6\times 4\,745.1}\right) = 361.$$

$$a_1' = \frac{3\times 3\,532.4}{2\times 195.6} = 27.1$$

$$a_1'' = \frac{3\left(4\,745.1 - \frac{2}{3}\times 5.85\times 219\right)}{2\times 219} = 26.7$$

$$a_1 = \frac{27.1 + 26.7}{2} = 26.9$$

$$t_L = 26.9 \text{ ft.} \qquad t_0 = 29.75 + (26.9 - 23.9) = 32.75 \text{ ft.}$$

Substituting these values in the equation, we have

$$29\,500 = \frac{1.11\times 83^2}{32.2}(26.9 - 32.75) - \frac{32\times 83^2\times 361}{135\,Q^2}(26.9^5 - 32.75^5)$$

$$= -1\,390 + \frac{588\,000}{Q^2}(32.75^5 - 26.9^5)$$

$$Q^2 = \frac{588\,000\,(32.75^5 - 26.9^5)}{30\,890} = 349\,000\,000$$

$$Q = 18\,700 \text{ cu. ft. per sec.}$$

---

* Fig. 41 is not drawn to scale.

The maximum velocity will be at the outlet, or at Station 80, and is given by

$$v_{max.} = \frac{18\,700}{3\,532.4} = 5.28 \text{ ft. per sec.}$$

This result shows that the method here presented gives maximum velocities, the magnitudes of which differ only slightly from those derived by the use of the uniform flow formulas. The remarks appended to the latter apply to this method without exception; in approximate calculations it should be used for short canals, to which the uniform flow formulas do not apply on account of the appreciable difference between the slope of the water and the slope of the canal bed.

11.—French Academy of Sciences' Formula.—In 1886, Count Ferdinand de Lesseps, Chief Engineer of the French Isthmian Canal Company, asked the Academy of Sciences to institute an investigation about the influence of tidal motion of the Pacific and Atlantic on the motion of the water in an open Panama canal. A committee, consisting of Messrs. Daubrée, Favé-Lalanne, de Jonquières, Boussinesq, and Bouquet de la Grye, was appointed by the Academy to answer the request of Count de Lesseps, and reported on the subject on May 31st, 1887. The report in its essence says:

1.—That the tidal variations at the Atlantic end of the proposed canal are so small as to be neglected.

2.—That experience shows that in a canal communicating on the one end with a sea of variable level, on the other end with a lake of constant level, the amplitude of the tidal curve diminishes uniformly from the sea to the lake, and further that the retardation of the tide is proportional to the distance, that, therefore:

If Y = half amplitude of the tide,
l = length of the canal,
w = velocity of propagation of the tides, and
t = time, in seconds, measured from the instant of low water at the sea, and 24 lunar hours being equal to 2 η, then the level, y, with respect to the mean canal or sea level, at a distance, x, from the sea, will be

$$y = -Y\left(1 - \frac{x}{l}\right)\cos.\left(2\,t - \frac{x}{w}\right).$$

3.—That, in accordance with what has been observed on similar canals, particularly on the Suez Canal between Suez and the Bitter

Lakes, the velocity of propagation of the tide can be represented by the formula:

$$w = \sqrt{g\left(H + \frac{3}{2}\,y\right)} \pm K\,v,$$

where H = depth of canal below mean sea level,
v = velocity of current,
K = constant (0.4 at flood tide, 1.2 at ebb tide).

4.—That, from the levels which have been derived by the foregoing equations for any moment and for two mutually not too distant places, the velocity of the current for any moment may be computed by applying the formula

$$v = C\,\sqrt{R\,S}.*$$

The velocities should be computed for, say, every half hour of the cycle, and the results tabulated; from this table the maximum velocity can easily be pointed out.

In applying this method of computation to a canal connected to tidal seas at both ends, the influences of the individual tides on the water elevations should be treated separately and the results added, with due regard to the sign of the slopes. The velocities should be calculated on the basis of the resulting slopes.

In order to check this formula against the Cape Cod observations, let it be applied to Station 227 + 50, admitting for the velocity of wave propagation the approximative value of

$$w = \sqrt{g\,H} = \sqrt{32.2 \times 28.65} = 30.7 \text{ ft. per sec.}$$

The maximum velocity at Station 227+50 occurred at 7.00 A. M., and low water at Station 80 was at 5.35 A. M., and at Station 375 at 2.50 A. M.

Taking first the influence of the tide at Station 80, and considering that the lunar time is 0.97 × solar time (nearly):

For $x = 14\,750$ ft.

$$y = -5.4\left(1 - \frac{14\,750}{29\,500}\right)\cos.\frac{0.97\left(2 \times 85 \times 60 - \dfrac{14\,750}{30.7}\right)360^\circ}{86\,400}$$
$$= -5.4 \times 0.5 \times \cos.\ 39^\circ\ 20' = -2.09 \text{ ft.}$$

* As a matter of fact, the Committee, having in mind the special problem of the Panama Canal, suggested the formula, $v = 56.86\,\sqrt{R\,S} - 0.07$ (metric).

At a distance 1 000 ft. from this station, or for $x = 15\,750$ ft.,

$$y = -5.4\left(1 - \frac{15\,750}{29\,500}\right)\cos.\frac{0.97\left(2 \times 85 \times 60 - \frac{15\,750}{30.7}\right)360^\circ}{86\,400}$$

$$= -5.4 \times 0.466 \times \cos.\ 39^\circ\ 10' = -1.95 \text{ ft.}$$

The slope, $S, = \frac{2.09 - 1.95}{1\,000} = 0.00014.$

The influence of the tide at Station 375 will, by inspection of Figs. 29 and 30, increase this slope, the half amplitude being 3.05 ft.

For $x = 13\,750$ ft.

$$y = -3.05\left(1 - \frac{13\,750}{29\,500}\right)\cos.\frac{0.97\left(2 \times 250 \times 60 - \frac{13\,750}{30.7}\right)360^\circ}{86\,400}$$

$$= -3.05 \times 0.534 \times \cos.\ 119^\circ\ 30' = 0.802 \text{ ft.}$$

For $x = 14\,750$ ft.

$$y = -3.05\left(1 - \frac{14\,750}{29\,500}\right)\cos.\frac{0.97\left(2 \times 250 \times 60 - \frac{14\,750}{30.7}\right)360^\circ}{86\,400}$$

$$= -3.05 \times 0.5 \times \cos.\ 119^\circ\ 20' = 0.749 \text{ ft.}$$

The slope, $S, = \frac{0.802 - 0.749}{1\,000} = 0.000053.$

The resultant slope $= 0.00014 + 0.000053 = 0.000193.$

Substituting this in the Chezy formula, and applying Kutter's coefficient for C, we get

$$v = 83\sqrt{18.6 \times 0.000193} = 4.97 \text{ ft. per sec.,}$$

a result which checks the observed velocities very closely.

The method here described has been thoroughly discussed and criticized in a paper by Dr. C. Lely,* who finds, in connection with studies relating to the Panama Canal, that the formula can be considered only as fairly approximate, but by no means accurate, because it does not comply with the law of continuity.

Judging from the remarkably close result furnished by the formula of the French Academy of Sciences when applied to the Cape Cod Canal, one must conclude that this formula gives the best approximate method for predicting the numerical value of tidal currents in canals

* *Proceedings*, Amsterdam Academy of Sciences, April 26th, 1907.

connecting two tidal seas. The objections to it which can be raised, namely, that the assumption given under Paragraph 2 is not exact, because the amplitude of the tidal curve cannot be a linear function of the distance, and the method of computing the velocities by the application of a uniform flow formula, not being accurate, are not serious for maximum values. The inaccuracy of the formula is much more evident at low velocities. In other words, near the time of the reversal of the current, the velocities deduced by its use change direction with the reversal of the slope, though it is a fact that, for a considerable time after the slope inclination has changed, the current still flows in the original direction. For canals as long as to make $q\ L$ equal 0.55 or more, this formula should be used to evaluate the friction coefficient, f_0', to be used in the formulas derived by harmonic analysis.

Conclusions.

The writer has endeavored to give a complete treatise on the question of tidal currents in canals, so as to enable the reader to gain a clear idea of this complex phenomenon, and has indicated the methods which can be used in predicting the magnitudes of such currents for given conditions.

There is no doubt in the writer's mind that the Airy-Lévy formulas, derived by harmonic analysis, with the modification proposed by the writer, represent the only rational method for solving the hydrodynamic problems arising within a tidal canal. This method is fully treated under the heading "The Solution of the Problem by Harmonic Analysis." The formulas for the evaluation of the elevation of the water and the velocity of the current at any instant are given in Section 5, page 115, the proposed modification of the same to meet actual conditions is discussed in Section 6, page 120, and the numerical application of the method is shown in Section 7, page 121.

It is questionable, however, whether or not, in practice, the refinements and complicated numerical work connected with the harmonic analysis are justifiable. The principal assumptions on which all the formulas are necessarily based, namely, the prevalence of a uniform cross-section and a uniform friction coefficient, are never quite true in the practical case of a canal dug in earth; and both cross-section and friction coefficient are apt to change materially with time. A number of other factors influencing the motion of the water, such

as wind, drainage of the water-shed tributary to the canal, etc., cannot be taken into consideration at all, and, *a priori*, reduce all analytical results to the grade of a more or less fair approximation. In addition, it should be noted that the designer of a canal is interested chiefly in the maximum velocity which may occur under given tidal conditions, and the behavior of the current for other values has little interest for him.

It would appear, therefore, that the approximate methods of calculation shown in this paper should be used for the practical evaluation of the velocities in canals subjected to tidal influences.

Three such methods have been suggested herein: In Section 9, pages 132-133, Bazin's, Eytelwein's, and Kutter's formulas relating to uniform flow are given. The formulas based on permanent but non-uniform flow are treated in Section 10, page 135, and those recommended by the French Academy of Sciences, *i. e.*, taking the velocity of propagation of the tidal wave into consideration, are shown in Section 11, page 138.

In the detailed discussion of these different formulas, the writer has shown that none of them can be adopted for general application, but that each of these approximate methods is best suited for certain distinct classes of canals, and, therefore, they should be used with caution.

The numerical results obtained by applying the approximate methods to the Cape Cod Canal would indicate that the formulas for uniform flow will give good results for a canal having the same characteristic features, both as to design and physical composition of channel bed. The approximate frictional factor to be used for a canal of this type is 0.03 in Kutter's formula. For other types of canals no such comparative statement can be made, because of the small number of existing tidal canals and the lack of accurate information in regard to the velocities.

The writer believes, however, that the results obtained by the application of the rational method (which are as accurate as can be at the present status of the science) furnish excellent means to determine the limitation of the use of the three practical methods presented. Therefore, taking these results as a measure of accuracy, the writer makes the following recommendations:

(*a*).—For tidal canals, the length, cross-sectional dimensions, and frictional characteristics of which are such as to make $q\ L$ less than 0.35, use the formulas for permanent non-uniform flow.

(*b*).—For canals where $q\ L$ lies between 0.35 and 0.55, use the formulas for uniform flow.

(*c*).—For canals where $q\ L$ is greater than 0.55, use the formulas of the French Academy of Sciences.

In $q\ L$, L is the length of the canal, in feet, and q has the value given in Equation (*8*), representing the influences of the form and area of the cross-section and the frictional properties of the channel bed. This value, q, being dependent on a great number of variable factors, cannot be represented by a diagram of general applicability, and should be evaluated in each individual case. To overcome the ambiguity caused by the fact that q is also dependent on the velocity to a certain extent, as a rule, a few trial calculations, as explained on page 120, will be necessary to arrive at the correct governing value of the product, $q\ L$.

Applying the proposed criteria to canals of the type of the Cape Cod Canal, *i. e.*, having nearly the same cross-section and the same consistency of channel bed, and being subjected to tidal differences of about the same magnitude, it can be stated that, for predicting the maximum velocities in such uniform canals:

If they are not longer than about 5 miles, use the formula for permanent non-uniform flow;

If their length is between 5 and $8\frac{1}{2}$ miles, use the formulas for uniform flow (Bazin, Kutter, etc.);

If they are longer than $8\frac{1}{2}$ miles, use the formulas of the French Academy of Sciences.

DISCUSSION

Mr. Herschel.

CLEMENS HERSCHEL,* PAST-PRESIDENT, AM. SOC. C. E. (by letter). —To construct and turn over for operation a public work that has been in contemplation for 300 years, assuredly makes its chief engineer deserving of recognition and praise from the Profession and from the general public; but when to this is added a carefully written account of the history of that work, the details of its construction, and an engineering treatise on the underlying principles of its design and operation, the paper becomes a notable one for insertion in the *Transactions* of the Society, and deserving the thanks of its members for the work done in gathering the data on which the treatise is founded, and in writing the paper itself. Especially is all this true in the present instance, when so little is generally known, even among engineers, of tidal action; and when the paper may be expected at last to put a quietus on a delusion than which the pursuit of witchcraft was no more brainless; which is capable of doing continued enormous harm; and which is yet active among us, as will be shown.

These preliminary remarks will sufficiently show the spirit in which the writer approaches the task of discussing this paper, and may counteract any notion of his intending it to be mere criticism. The very fact of a high value set on the paper, has engendered in the writer a wish to have it as near perfect as discussion can make it, and has caused him to contribute what he can to that end.

Perhaps the most curious historical statement in the paper is the following on page 12: "This contribution of General Foster's changed completely the whole character of the enterprise, because, after the publication of his report, a canal with locks was not again considered."

The writer, who was active in Cape Cod Canal matters in 1870, and for 40 years thereafter, desires to point out that, though locks could not have been seriously considered by engineers, after 1870, laymen and competitive interests even yet speak of locks in connection with the Cape Cod Canal. The charter of 1870, antedating General Foster's report, contemplated a Cape Cod Canal without locks, and no charter granted since then ever was or yet is absolutely secure from having locks foisted upon it. The trouble that prejudice and ignorance are capable of accomplishing in the affairs of men is not so easily disposed of as the paper before us would cause us to believe; and now for the proof.

The following is from the letter-press book of the writer; and will the reader please note the date:

* New York City.

Mr. Herschel.

"BOSTON, January 18, 1870.

"THOMAS LAMB, ESQ.,

"Prest. N. E. National Bank,

"Sir,

"Agreeable to your desire, I have examined,—to no great degree of completeness however,—into the subject of a Cape Cod Ship Canal. I have no doubt that a single system of locks, as you suggest, is sufficient to operate such a canal and is preferable, in more than one respect, to two systems of locks,—one at each end,—and I should think such single lock ought to be placed over near the Buzzards Bay end of the Canal.

"After due reflection and some investigation, however, I think the Committee appointed by the Senate and House in 1860, in their Report of November 5, 1862, make a sound suggestion, when they speak of the propriety of investigating,—which has never been done,—whether a canal without locks could not be made; by making the same, say, 20 miles long, instead of 7¾ miles, as then surveyed.

"This would be doing in canal-building only what has been done on every Railroad that ever was built; also on ordinary canals, etc.; that is, increasing the length, to diminish the descent or fall per unit of length; and I find that such a canal twenty miles long need not have a greater surface velocity than 2.2 miles per hour, or bottom velocity so great as 3 ft. per second, at the times of the greatest difference of level between the two Bays.

"In case of a canal confined by artificial banks and having a current of 3 ft. per second flowing through it for a considerable length of time, this velocity would give good grounds for apprehensions, and it might be impracticable to make the banks firm enough to resist it; but with a canal excavated in the natural soil, below the level of the surrounding country as this would be, and having the maximum velocity only twice in 24 hours for a short time, the difficulties of the case can, I think, be boldly met and overcome by simple precautions or remedies, to enumerate and describe which would make this communication too long and elaborate. To give you an idea as to what rank such a velocity of current is entitled to, it may be stated that the same and greater velocities exist in multitudes of inlets found on every coast where the tide flows in and out. High waters and freshets in rivers have velocities as great as 12 ft. per second and over.

"I am satisfied that the cost of a longer canal without locks would be about the same as that of the short one with locks and artificial harbors; that it would cost much less to operate, and would be more used by seamen. Further, we are in the one case compelled to accept of some inferior harbors or sites for such, as termini of the canal, while in the other, we can select good harbors to be the vestibules, if I may so call them, to either end of the canal; and in this one item alone I am satisfied that millions could be saved.

"Very respectfully yours,

"CLEMENS HERSCHEL,

"Civil Engineer."

Mr. Herschel.

The "sound suggestion" referred to is the one quoted on page 11 of the paper (page 54 in the original report), beginning:

"It may not be unreasonable to suppose that some method of avoiding the force of the currents might be discovered without the cost and delay of using locks and gates."

As stated, the letter above quoted was written after only a very brief consideration of the subject, and errs on the side of safety, in the endeavor to keep at a minimum those terrible currents, which, in the popular belief of the day (to some minds yet existing), were going to work havoc with the geography of the "Old Bay State."

General Foster's report is dated May 10th, 1870. The Resolve of the Massachusetts General Court, in consequence of which General Foster's views were called for by a Congressional Committee and by the Chief of Engineers, is dated April 2d, 1870. The charter of 1870 is dated February 26th, 1870.

To the best of the writer's recollection, he brought to General Foster's attention, and advocated directly to him, the advantages of an open sea-level canal; a form of canal, which, at that time, and often since, somehow or other, has been unpopular; but which, in most or all cases of a tidal canal, is the best form, and will no doubt hereafter generally prevail.

It was General Foster's merit, however, to cut loose from the unfounded fears of a short-line Cape Cod Canal, one built on a route only 8 miles long, and, by computations, to show that the surface slope to be expected on such a canal, and their consequently engendered velocities, need not be feared as rendering ordinary steam navigation impracticable. His report also points out, in conclusion:

"* * * that it may, and should be made an open canal, without locks, in order to accommodate the greatest number of vessels that may present themselves for passage, and at the same time keep the canal clear of ice" (and sand, he might have added). That he adds, "* * * that a breakwater at the eastern terminus is necessary as a protection to the mouth of the canal, and important as a harbor of refuge for vessels navigating the bay" (meaning a breakwater, at right angles to the centre line of the canal, out in deep water of Sandwich Bay); and: "It is also proposed to provide, as a precaution against the effects of very high tides, guard gates at each end of the canal, which may be closed in an emergency to avert danger to the Canal, but which are ordinarily to be open"; with an allowance of about $3\frac{1}{2}$ million dollars for breakwater and guard gates; that he adds all this is only "throwing a tub to the whale" of popular clap-trap. But there was and is one engineer, at least, who never could stomach prejudice and self-assertive ignorance, come what may, and he kept up the contest for an open sea-level canal without breakwaters, or guard gates, or any other gates, as will presently appear, until, due to the enterprise

of August Belmont and others, he had the great pleasure of navigating just such a canal through Cape Cod in 1914. Mr. Herschel.

Before General Foster's day and since, many engineers have considered it politic, expedient, leading to profit, what not, to defer to the cry of the populace.

General Foster could hardly avoid estimating for the breakwater, because the Resolve above referred to, specifically called for a breakwater; and the demand for safeguarding the canal from those terrible currents, was very strong.

General Foster left Boston in 1872, and died in 1874, and thus was prevented from actively furthering the construction of any Cape Cod Canal; as he undoubtedly would have done, had he lived.

There were, of course, certain "interests" that did not want the canal built. The freight traffic between New York and Boston, by railroad; by the mixed railroad and steamboat lines; and by one "outside" line, was very comfortably fixed; and ready to cause others to let well enough alone. It was easy to slip a clause into the several charters, looking to the construction of locks in, or enabling certain Boards to force them upon the canal; all of which would with almost certainty cause that charter eventually to lapse. For a canal, frozen up every winter, built to facilitate winter traffic around the Cape, is palpably an absurdity; let alone other most serious disabilities of such a canal, as will presently appear.

Does any one think that these absurdities, and arguments for having them come into being, are extinct? If so let him turn to page 100 of the "First Annual Report of the Commission on Waterways and Public Lands", of Massachusetts, for 1916 (issued as this is written), and read as follows: "Requisitions * * * pending a determination of the question of a lock, tidal gates or other device for controlling the current of the canal, and of other matters." Or let him scan the much abused 1917 United States River and Harbor Bill, the Section relating to the purchase and making of a free channel (as it ought to be), by the United States, of the "Waterway connecting Buzzards Bay and Cape Cod Bay, Massachusetts", "with or without a guard lock."

At variance with the statement on page 12 of the paper, the writer's direct professional relations with the Cape Cod Ship Canal Company chartered in 1870, did not begin until 1878. But in one way or another he had such relations with men and corporations interested in the project in 1870, and thereafter. In this way was written the 1878 report (forty closely printed pages), on the Cape Cod Ship Canal, and, in 1884, the Company chartered in 1883 reprinted it with additions. It is to be found in various libraries, and is believed to have been useful as a source of information on the subject ever since.

This report is too voluminous to quote. It advocated for the first time an open sea-level canal, without locks or gates of any kind, no

Mr. Herschel.

harbor-producing breakwaters, two parallel jetties to extend the banks of the canal into the deep water of Sandwich Bay; the very type of canal built in 1909, more than 30 years later. One feature of the report, of course now out of date, but presumably of value as showing a method applicable in other such investigations, was a determination of how much the carrying of coal around the Cape, rather than landing it south of the Cape, had actually cost for the 10 years, 1868-1877, both inclusive. This was done by looking over the books of several coal dealers each in Boston, Salem, and Lynn, and comparing these freight rates paid, with those paid by several dealers each in Providence, Fall River, and Newport; doing this for every month of the 120 months considered; adjusting the difference found in proportion to the proportional shipments of coal during the several months in the year (more per month in the summer than in the dangerous winter months); and doing all this twice, once with New York (Rondout), as the shipping port; and, again, as coming from Philadelphia. Two months, in the 10 years, this difference was $1 per ton; and for 2 months it was nothing.

The true average difference was 57 cents per ton, shipping from Philadelphia, and 48 cents shipping from New York. For purposes of the report, and attempting to foretell what that difference would average for the next 10 years, 30 cents was adopted. And attention was called to the effect of shipping in tows of barges, etc., etc. Since then, the shipments of coal around the Cape have increased about four-fold.

To justify a conviction that those currents that have been spoken of would not saw the earth in halves, beginning with the slight notch cut on its surface by dredges, and then allowing the tides to act, the author of the 1878 report also wrote an article,* more particularly for engineers' reading, entitled "On the Erosive and Abrading Power of Water."

It may now be thought that this discussion exaggerates the fear held by some, and studiously engendered in others, as to the effect of the expected canal currents on the Canal. To illustrate what it was, the writer remembers that a colleague and sort of running mate, half surveyor, half astronomer, of "Professor" Mitchell, referred to on page 10, publicly denounced the project of an open canal, by saying that he would "as soon think of turning loose Niagara across the State of New York;" very effective as a piece of rhetoric, no doubt, but less valuable as an item of engineering judgment or knowledge. In February, 1901, the writer was engaged to argue, and did argue, before the Harbor Commissioners of Massachusetts, in favor of allowing the canal to be built as an open sea-level canal. Other occasions for the writer's activities in favor of an open sea-level canal at

* *Journal*, Franklin Inst., May–July, 1878.

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Cape Cod than those mentioned came to pass during the 40 years, 1870 to 1910, but it will be needless to refer to them.

Passing now to Part II, The Hydraulics of the Cape Cod Canal, a general review of the situation must bring out, in strong relief, the usual situation with regard to the practical hydraulic problem of the flow of water in artificial channels. Formulas based on data observed in any actually built channel or channels are only approximately applicable to other such channels, because none of the channels mentioned is ever mathematically uniform, or regular, or exactly similar in construction; and when based on data procured from specially constructed regular channels, is only approximately applicable (by reason of irregularities found in them) to channels as they are found in actual practice.

A classical example of the first part of the stated proposition is that furnished by a comparison of the discharge of the Sudbury Conduit, built 1873-1883, supplying Boston, and that of the last-built Croton Aqueduct, 1887-1895, supplying New York City:

"These two works were built under practically identical leadership, and under precisely similar circumstances and conditions, the one immediately after the other. The same engineers, the same class of materials, identical methods of construction, distinguish them. Most excellently conducted gaugings or experiments of discharge were made upon the Sudbury Conduit, and its formula of discharge was computed when it was new. Yet when such measurements were repeated on the newly completed Croton Conduit, only 94.5% of the expected results were attained."*

In a measure, this weakness, or inexactness of hydraulic formulas, is recognized throughout the paper; as is also the fact that, in computing for a navigation canal, an exact mathematical evaluation of expected currents, is of no practical value. "The average skipper cares very little whether he has to tow (or run) against 3.3, or 3.6, or 3.7, (or 4 or 5) knots per hour, for a distance of 8 miles."† The several given methods of computing probable future velocities in a tidal canal, however, are of value in giving engineers their choice of which they will use, and of catering to the most fastidious. For those who like computations to the third decimal figure, based on data that are not exact to one decimal figure, some of the formulas referred to are of the kind they like. Also of value in demonstrating that the results of computation by any and all of the formulas given, differ only within reasonable limits. If formulas for uniform flow in masonry conduits, as above mentioned, differ by 5.5%, what can be expected of formulas that deal with tidal flows in irregular excavations in sand and gravel? In the first case, million gallons per 24 hours

* See p. 77, "115 Experiments", by the writer. John Wiley and Sons, 1897.

† *Journal*, Franklin Inst., May–July, 1878.

Mr. Herschel.

actually discharged into reservoirs are sought after; while in the latter it is only a question whether the sides of the canal can be made current-resisting, and the canal itself navigable, between the limits of "slack water", and a maximum to be expected velocity.

A word more as to navigability in currents of different value. Sailing vessels in tidal canals can be left out of consideration. They and small boats can, if permitted, drift through with a favorable tide, at the proper hours, or hire "tows." Steamers take care of themselves, or may be assisted through. As a matter of fact, the channels of commerce of the world abound in swift water currents, as shown in a record* of such, prepared by the late President, Am. Soc. C. E., Elmer L. Corthell. They vary, materially: 7.5 knots in the St. Lawrence River; 10 knots in Portland Firth; 13 knots in the Gironde, France; and a multitude of others. On rapids in the Rhine, Elbe, and Rhone Rivers, (more than 9 knots), cable towing has been in use for many years. It was proposed, if found necessary, for the Corinth sea-level canal, but was not found necessary.

An along-shore fisherman, or navigator, or draw-tender, who has known the Monument River, man and boy, for 30 or 40 years, is, of course, very much astonished, or may even feel outraged, on viewing its ancient sea channel, at an hour, perhaps, when there is a 4-knot or even a 5-knot current to contend with, to see the tide now running there. Had he fished, navigated, or viewed the Harlem River, we will say, between the Hudson and the East Rivers, the same length of time, he would think nothing of it.

The investigations of Mr. Corthell and of his predecessors showed clearly that uniformity of cross-section, at least in a hydraulic sense, (uniform mean hydraulic radius, and uniform depth, also a good alignment), were the great elements making for good navigability. Nothing so sensitive as a constriction, even only a slight one, in a tidal channel.† Any one who has worked in a tidal stream, with, say, a 10-ft. tide, and has seen a roaring tideway quiet down to slack water inside of about 2 min. by the watch, only to turn around and become a roaring tideway in the other direction inside of another 2 min., will ever forget it, and what it teaches. All changes of cross-section must be made very, very gradually, as has been spoken of in the paper under discussion.

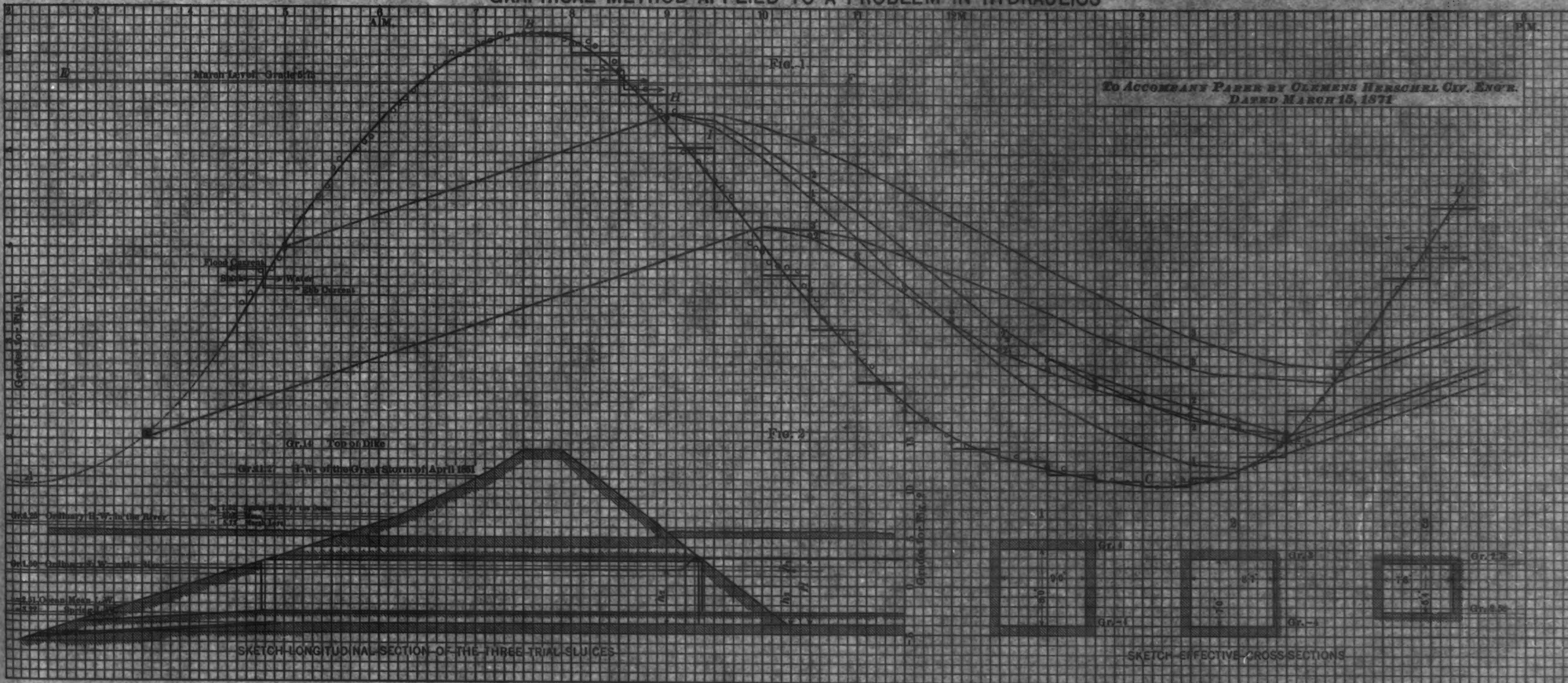
The same considerations also make little better than arrant nonsense of the constantly recurring proposition, sometimes in very high places, to put "guard-gates", "tide-locks", what not, into an open sea-level canal, to be used only occasionally. It would be impossible or impracticable to make such guard-gates or locks, of an equal

* *Memoires,* Société des Ingénieurs Civils de France, 1906.

† See the paper "On Waves of Translation that Emanate from a Submerged Orifice, Together with an Examination of the Feasibility of the Proposed Baie Verte Canal", *Transactions,* Am. Soc. C. E. (July, 1875), Vol. IV, p. 185, by the writer.

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1403.
HERSCHEL ON
CAPE COD CANAL.

GRAPHICAL METHOD APPLIED TO A PROBLEM IN HYDRAULICS



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hydraulic radius and depth and area as the main canal; or, if so made, to connect it to the main channel gradually, and by sufficiently slow degrees. The consequence would be that the tidal wave would at once pile up at such a constriction, making there a point of dangerous navigation, back and forth, as just referred to. And as for "controlling" the currents in such a channel, by the operation of such gates, some of the time, that again is wholly impracticable. A "bear-trap" dam at each end of the canal might do it, but would not be navigable; and no "gates" of any kind could with safety to the gates be operated in such a canal, with or against the current. Mr. Herschel.

To the writer, it appears that "the excellent Homer nods" on page 104. Times of high water at Stations 410 and 35, surely do not control, nor are they in the main the results of, velocity of wave propagation in the canal. Times of high water at either end of the canal are almost exclusively dependent only on times of high water in the ocean bays at either end, and are very little influenced by wave propagation, positive or negative, both of which take place, through the canal. The writer has great respect for the formula given on page 103, $w = \sqrt{g\ H}$, having himself by experiment* tested it. In his opinion, it no doubt acts in full force and continuously from both ends of the Cape Cod Canal, though, on only 8 miles of length, it has small opportunity for producing any notable or material effects.

Again, on page 121, picking out "a stretch of uniform or nearly uniform cross-section", "to test the validity of the equations supposed to furnish the values for the elevation of the water and velocity at any section and time" will not overcome the difficulties of the case. These elevations and times, in the case of a tidal canal, operated, as it is, by the great outside ocean tides, are functions of every foot of the length and depth of that canal, and of its varying cross-sections, in the open and under bridges, etc., from ocean to ocean.

In spite of what has above been said as to methods of computing, the writer ventures to add what would have been, or would be, his method of computation, had he been, or were he, called on to compute water heights and velocities in a long sea-level canal. It calls for no formulas except $v = C\sqrt{R\ S}$ and $w = \sqrt{g\ H}$, and may be gleaned or inferred from an article of his, entitled: "On the Solution, Mainly by the Aid of Graphical Construction, of a Problem in Practical Hydraulics."† The diagram accompanying this article is herewith reproduced as Plate I.

It will be noticed that the method referred to is based on assuming that the ocean tide rises, not in the form of a sine curve (when plotted with respect to times), but in steps following a sine curve

* See the paper "On Waves of Translation that Emanate from a Submerged Orifice, Together with an Examination of the Feasibility of the Proposed Baie Verte Canal", *Transactions*, Am. Soc. C. E. (July, 1875), Vol. IV, p. 185, by the writer.

† *Journal*, Franklin Inst., 1871-72, pp. 105 and 181.

Mr. Herschel.

in outline. Then, by taking the "tread" of these steps as short as may be desired, any desired degree of coincidence with a sine curve rise of tide may be attained.

There are now in existence and recorded in engineering literature, so far as the writer knows, just five open sea-level canals:

Suez Canal,
Corinth Canal,
East Bay Neck Ship Canal,
Pamlico Sound-Beaufort Canal,
Cape Cod Canal.

The "Kiel Canal" would be another, only that, for purposes of marsh drainage along its line, it was necessary to put in lock gates at each end, and keep the canal level at mean level of the sea.

The first three named have been discussed by the writer,* together with the, at that time, only projected Panama Canal. So was the Cape Cod Canal at that time, only a projected canal; likely to remain a mere project for another century or so, by all the unkind (as we will call them) things said of open sea-level canals in reports on the Panama Canal. A contention in favor of an open sea-level canal at Cape Cod can be read between the lines of much that the writer then wrote about a sea-level canal at Panama; and proof direct could be given, how some of the intended decrying of open sea-level canals, intended to bolster up the agitation in favor of a lock canal at Panama, was designedly brought to naught, as a protection to a future building of an open sea-level canal at Cape Cod.

That the Panama Canal was built as a lock-canal should disturb no one. The hydraulic questions involved were admittedly never properly considered, for the reason, as given, that "the limited time available to the Board [of Consulting Engineers] has not permitted the full consideration of this question, which is desirable;" a polite shirking of duty in an important matter, probably without parallel in engineering history.

It has now been given to Mr. Parsons to demonstrate by physical facts, and by the first comprehensive treatise on sea-level canals written in the English language, accompanying a work of which he was the Chief Engineer, that sea-level canals are amenable to common sense and to engineering computations. This, and the fact that he was one of the three American engineers on the Board of Consulting Engineers (William H. Burr, M. Am. Soc. C. E., and Gen. George W. Davis were the two others) who, in 1905, did not join in the hue and cry for a lock canal at Panama, but voted with the majority of the Board composed of these three, together with all the well-informed British, French, Dutch, and German engineers selected

* *Transactions*, Am. Soc. C. E. (June, 1906), Vol. LVI, p. 206.

Mr. Herschel.

by their respective Governments as life-long experts on the questions involved, make up a record of which any hydraulic engineer may be proud, and to which the writer believes time will only add high repute and renown.

(Mr. Herschel attended the meeting at which Mr. Parsons' paper was presented; he did not read the foregoing written discussion, but addressed the Society as follows:)

The speaker has written a discussion on this paper, but, instead of reading it, he will consider the Cape Cod Canal from another angle than that from which he viewed it in the foregoing discussion.

This paper is very important, for more reasons than one. If one looks at a map of the United States and follows carefully the inland, or nearly inland, waterways along the east coast, one will soon come to the conclusion that it would not require much of an engineering undertaking to enable a vessel to go from Massachusetts Bay to the Gulf of Mexico by an inside passage. It is almost, or wholly, possible now for very small craft, and one can see clearly that, before a hundred years have gone by—or possibly before 20 years—that route will be open for vessels of possibly 25 or 30 ft. draft.

Now, this Cape Cod Canal may be considered as the first link in that chain of communications, important for freight traffic, and of the greatest importance as a measure of national defence.

There is in existence an association, called the Atlantic Deeper Waterways Association, which has as its object the construction of an inside waterway all along the eastern coast of the United States, and for the last 10 years it has advocated this project. A Member of Congress, J. Hampton Moore, seems to have made this his life work, and every year this association has excursions over and inspections of parts of this waterway. Thus far, that seems to have been the only method of agitating the subject. It is hoped that in the course of time the prosecution of the public works of the United States will be carried on in a more rational way than by pulling and hauling and junketing and picnicking. All are agitating for such improved and more rational ways of selecting needed public works, and there is good reason for believing that we are on the very threshold of a rational method of making such a selection.

It may not have been noticed by all engineers, but the last river and harbor bill had incorporated with it a clause providing for the formation of what has been called the Waterways Commission. The speaker thinks that this clause has been so little noticed that he will venture to read what this waterways commission proposes to create and do. It has been enacted:

"That a commission, to be known as the Waterways Commission, consisting of seven members to be appointed by the President of the

Mr. Herschel.

United States, at least one of whom shall be chosen from the active or retired list of the Engineer Corps of the army, at least one of whom shall be an expert hydraulic engineer from civil life, is hereby created and authorized, under such rules and regulations as it may adopt, to bring into co-ordination and co-operation the engineering, scientific and constructive services, bureaus, boards and commissions of the several governmental departments of the United States and commissions created by Congress that relate to study, development or control of waterways and water resources and subjects related thereto, or to the development and regulation of interstate and foreign commerce, with a view to uniting such services in investigating, with respect to all watersheds in the United States, questions relating to the development, improvement, regulation and control of navigation as a part of interstate and foreign commerce, including therein the related questions of irrigation, drainage, forestry, arid and swamp land reclamation, clarification of streams, regulation of flow, control of floods, utilization of water power, prevention of soil erosion and waste, storage and conservation of water for agricultural, industrial, municipal and domestic uses, co-operation of railways and waterways, and promotion of terminal and transfer facilities, to secure the necessary data, and to formulate and report to Congress as early as practicable a comprehensive plan or plans for the development of waterways and the water resources of the United States for the purposes of navigation and for every useful purpose, and recommendations for the modification or discontinuance of any project herein or heretofore adopted."

Then follows a provision for pay and compensation of members, and other clauses which it is not necessary to read; but that gives one an idea, and the speaker thinks that all will agree with him that the intention of the draftsman was at least comprehensive, and that we may hope that some good will come of it.

Now, the Cape Cod Canal is a public work, completed by private enterprise, which is directly in line with this clause which has just been quoted, and it is to be hoped that this commission will continue the good work.

A right of way for a sea-level canal, without locks, from New York Harbor to the Delaware River has been promised by the State of New Jersey; the routes have been surveyed by the United States Engineers, and no difficulties have been found. There is a canal now from the Delaware River to Chesapeake Bay, and so on. There are several other small canals that could be widened and deepened. One of these, on the same line, is already a reconstructed sea-level canal; and, for such reasons as the speaker has given, he thinks this paper of great importance, and that the uses of the canal will grow as time goes along.

Mr. Thomson.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—The Society is much indebted to Mr. Parsons and Mr. Douglas for a well-prepared and well-presented paper on this interesting work, and Mr. Belmont and

* New York City.

his associates deserve the thanks of the entire country for carrying through a much needed public work as a private enterprise. Mr. Thomson.

It can easily be understood why it was necessary to economize on width, depth, etc., but, now that the enormous future value of this canal must be apparent, the Government should give every aid and encouragement to the efforts to obtain an ultimate depth of at least 40 or 45 ft., an ultimate width of at least 1 000 ft., and, eventually, instead of bridges, tunnels carrying traffic under the canal. These figures may sound fantastic, but they are not; although the speaker may be laughed at now for seeing "too much", he may also be laughed at by future generations for not seeing far enough.

The Atlantic Deeper Waterways system which Mr. Herschel has explained so well (the speaker has also been a member of the Deeper Waterways Association for some years) will eventually result in deep inland waterways from Boston to Florida. It is true that, at present, the Hon. J. Hampton Moore only contemplates a depth of 12 ft., or less, in some places, but, as soon as these depths are obtained, greater and greater depths will be insisted on. A glance at a map of the Atlantic Coast will astonish most people by showing how comparatively few links of canals will be required to complete the inland waterways from Boston to Florida and from New York to the Great Lakes.

The speaker takes this opportunity to "report progress" on his project for "A Really Greater New York", which will afford the most important link in this chain of inland waterways. After working nearly every day for six years and writing repeatedly to three Governors of New York State and two of New Jersey, as well as to two Mayors of New York City, and many prominent men, and addressing this Society on more than one occasion, the speaker is delighted to state that real progress has been made, for Governor Whitman and Governor Edge have appointed a joint board to make this harbor proposition an interstate affair. These men are: Messrs. William R. Willcox, Chairman, J. Spencer Smith, Vice-Chairman, E. H. Outerbridge, Arthur Curtiss James, De Witt Buskirk, and Frank R. Ford, with Gen. Goethals for Chief Consulting Engineer.

The speaker has photographs of a suction-dredge, having a discharge pipe 42 in. in diameter, now used in Egypt. It was designed by A. W. Robinson, M. Am. Soc. C. E., undoubtedly the greatest authority on dredges. The actual work of this dredge has been 50 000 cu. yd. per day, at a cost of less than 1 cent per yd.

When the "Really Greater New York" project is started, we will probably have several of these machines and will be glad to lend them to the Cape Cod Canal—between times—to widen and deepen the canal as will be required.

The speaker does not think it safe to assume that there will be no danger from the teredo at Cape Cod, for he was called in a few years

Mr. Thomson. ago to examine a bridge at Fall River, where one pier had settled 2 ft. over night. This condition was due to the teredo destroying the piles within two years of the time the bridge was built.

Many think that the teredo attacks only at or near the surface of the water. In this case, however, the piles had been cut off 45 ft. below the surface, and the pier (granite face and concrete backing) had been built on a 4-ft. grillage, as the grillage was sunk to the tops of the piles. The speaker saw one of the pile heads which was brought up and split open, showing both live teredo and live limnoria hard at work.

The pier had settled 2 ft. at one end, so a coffer-dam was built around it, concrete was forced under the grillage, and the bridge seat was restored to its proper level, with satisfactory results.

New York Harbor is now safe from the ravages of the teredo, but when the speaker's project for "A Really Greater New York", which involves great trunk sewers down the present bed of the East River, also through New Jersey to a point some 20 miles away from Sandy Hook, is accomplished, the rivers and bays of New York City will be safe to fish and swim in, and the piles of the wharves will have to be protected against the teredo.

Mr. Parsons. WILLIAM BARCLAY PARSONS,* M. AM. SOC. C. E. (by letter).—The discussion by Clemens Herschel, Past-President, Am. Soc. C. E., adds much interesting matter to the paper, especially in bringing out his own constant and consistent efforts in advocating a sea-level canal and combating the unexamined fear of a destructive tidal current. The writer's reference to Gen. Foster's contribution is possibly too broad, because, as Mr. Herschel points out, there are certain laymen who, even to this day, are prejudiced against an unlocked canal, but, after Gen. Foster's report, no serious consideration was ever given to a lock by those earnestly interested in the matter, as far as the writer has been able to ascertain.

The writer is distinctly pleased with the clear and forceful manner in which Mr. Herschel brings out the folly of any attempt at guard-gates or any means of partial control of tidal currents. A canal must be either a lock or an open canal. Any contraction caused by a lock or guard-gates will produce local difficulties and such a local increase in current velocity as to defeat the idea of a compromise. In the Kaiser Wilhelm or "Kiel" Canal, not lock-gates but actual locks were constructed, on account of marsh drainage, as Mr. Herschel points out. It was expected that, on ordinary neap tides and during a part perhaps of all tides, the gates might remain open and the canal be a tidal waterway. It was at once found, however, that a local current developed, due to the contraction of the area of the cross-

* New York City.

section, and that this was inconvenient, although the tidal current in the whole canal was negligible. On this account the gates are always kept closed, and the canal is used at all times as a lock canal. Mr. Parsons.

If page 104 of the paper is read, it will be seen that Mr. Herschel, in his discussion, attributes to the writer a statement which he never made, namely, that times of high water at Stations 410 and 35 are, in the main, the result of the velocity of wave propagation in the canal. On that page it is stated that "the velocity of wave propagation" is "a function of the time interval between the high-water points of the two ends," and, further, that "high water along the canal at the consecutive stations will be reached in the time interval between the high waters at Buzzards Bay and Cape Cod Bay." The paper deals with a tidal wave passing through the canal, and, by "velocity of wave propagation" is understood the speed with which the crest of this wave travels. In canals where tidal action occurs at one end only this velocity is mainly a function of the depth of the canal. In the Cape Cod Canal, where there are considerable tides at each end, the velocity of wave propagation is dependent on the phase and amplitude differences of the actuating tides.

Messrs. Herschel and Thomson are right in advocating a wider and deeper canal. The present limitations were imposed by financial necessities. Although, in a larger canal, the tidal current would be increased, ease in navigation would also be increased. The effective current is that which acts on the vessel, and this rate is increased according as the cross-section of the vessel diminishes the area of the canal. It is obvious that, with the same vessel, if the canal cross-section is doubled, the vessel will take up only one-half as much, and, in round figures, the effective current will be reduced one-half. In constructing the enlarged canal, Mr. Thomson would find that suction-dredges are not adapted to handle the material found on Cape Cod.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1404

ADDRESS AT THE ANNUAL MEETING, JANUARY 16TH, 1918.

BY GEORGE H. PEGRAM, PRESIDENT, AM. SOC. C. E.

The abandonment of the Annual Convention last June on account of the War, moved the Board of Direction to request your President to deliver his Annual Address at this meeting.

Realizing the demands upon your time for other matters, I have viewed the action of the Board as largely a matter of courtesy, which I deeply appreciate, and shall endeavor to be brief.

The great world-war is imposing supreme tests of the virtues, faults, and philosophies of all peoples, and their industrial, economic, financial, and political organizations will undergo great changes.

In this reformation there is the greatest hope for the engineer; and as he has always been the keystone in the arch of war, he may become the keystone in the arch of peace.

Practically all the world's great military commanders have been engineers: Cæsar, Bonaparte, Wellington, in the past, and, in the present war, Joffre, Haig, and Hindenburg.

In our own country, many of the men who gained prominence during the Revolutionary War were surveyors, that being the general work of the engineer of that day.

Ira Allen, of Vermont, one of the ablest diplomats and financiers of his time, was a surveyor, serving for a number of years as Surveyor-General of the State. He was one of the first to propose building the canal to connect Lake Champlain with the St. Lawrence River.

General Stark, the hero of the Battle of Bennington, was a mill-wright. General Wayne, like his great commander, General Washing-

ton, was a surveyor, and, at the beginning of the Revolution, was surveying in Ohio. In our Civil War, the engineer was similarly prominent, and this was emphasized among the officers drawn from civil life.

War involves many engineering operations; in fact, at the present time, it has become an engineering science; so that we might expect such distinction, were it not that higher qualities than mere professional technique are required in generalship, which demands the organization and control of masses of men, and administrative ability of the very highest order. The engineer has so generally responded to these higher requirements in times of war, when the best services must be had, that we may reasonably ask why greater use is not made of his administrative qualities in times of peace.

Nearly three years ago, our engineers began the agitation of preparedness for war. In March, 1915, a committee was appointed whose work resulted in the law authorizing the Engineer Officers Reserve Corps, by which civilian engineers are made part of the army; groups of engineers in different parts of the country provided military lectures, preparedness parades, and other movements anticipating the possibility of our being drawn into the war.

The Administration at Washington, in appreciation of the services which might be rendered by engineers, wisely called them into council, resulting in the formation of the Naval Advisory Board, the Council of National Defence, and the Bureau of Scientific Research, in all of which our most skilled and prominent engineers have been from the first, and are now, unselfishly devoting their services to the needs of the country, without remunerative return, even for their expenditures; and our Society, as you know, offered its services to the President immediately upon the declaration of war, and has responded to every call.

An engineer, Mr. Herbert C. Hoover, has been selected for possibly the greatest administrative post, the head of the United States Food Administration, and to this Administration we have given the free use of our former home which cost a half million dollars.

In spite of the fact that but 11.5% of our membership is within the draft age limit, 11.0% is now in the army, and the number is rapidly increasing. Men of mature age, without previous military training, and whose prominence in the Profession would entitle them

to entire immunity, have gone to the front. The names of Parsons, Wilgus, Bensel, Buck, and Hodge, among my intimates in New York, come to mind, and of course there are hosts of others.

The glorious record of the first appearance of the American soldier on the firing line of Europe fills us with pride. Our engineers were aiding Byng's advance on Cambrai with the construction of the military railroad, and progressed so fast that a recession of the line brought them on the front, when they dropped their picks and shovels and grasped guns from the wounded and dead and joined the fighting forces.

The Engineering Mission from the United States to Russia, consisting of John F. Stevens, John E. Greiner, W. S. Darling, George Gibbs, and Henry Miller, has performed a task which, despite the political revolution there, must bear great future fruit; for Russia, next to China, has the greatest undeveloped resources of any country in the world, and hereafter the field of the engineer, like the present field of the soldier, will be less limited by national boundaries.

Engineering is sometimes claimed to be a modern profession because the present appellation is of comparatively recent adoption. The practice, however, is of such ancient origin that, in considering its relations to the community, we are obliged to view its history. Most of our tools—the dowel, drill, chisel, wedge, screw, pulley, file, and saw—were used by the ancients.

No works of modern times compare in magnitude with those of the ancients. Consider a reservoir, to impound the waters of the Nile, covering an area of 150 sq. miles, with a dam 30 ft. high and 13 miles long. The pyramids of Gizeh, constructed more than 5 000 years ago, had granite blocks which were 5 ft. square and 30 ft. long, and were transported 500 miles. One of the temples of Memphis was built of stones which were 13 ft. square and 65 ft. long, and laid with close joints. The Appian Way from Rome to Capua was so well built that after a thousand years its roadway was in perfect condition, and, even now, after two thousand years, with slight repairs, is in use. The modern engineer would question the possibility of such work, without these great examples.

If one could imagine cessation of life on this continent, and our works subjected to the destructive forces of time and Nature for a thousand years, what evidences of civilization would remain?

Probably the most surprising and interesting of the older examples of engineering are the inventions of Leonardo da Vinci, as shown in his sketches. He seems to have lacked nothing but the application of mechanical power to produce most of the typical machines of the present day. The bellows-blast, jig-saw, lathe, rolling mill, printing press, file-cutting machine, trip-hammer, sprocket-chain, water-wheel, boring machine, rapid-fire gun, spinning machine, side-wheel boat, flying machine, etc., and I commend to your notice the work of Franz M. Feldhaus on "Leonardo da Vinci as Engineer and Inventor", published in 1913. In this work is shown an apparatus of cylinder, piston, and valve by which Leonardo determined the relative volumes of steam and water; also an atmospheric engine, consisting of cylinder, piston, and valve, by which reciprocal motion was produced.

The pity is that, in all his machinery requiring mechanical power, and in spite of his experiments hinting at its application, he still was limited to muscular effort, and it was not until the invention of Watt that the mystic wand of mechanical power initiated a transformation of the world and made a radical change in the organization of man.

We look in vain for the application of mechanical power by the ancients, whose works seem almost impossible without its assumption, but the stone reliefs showing the movement of large weights by manual power indicate that probably the other did not exist.

An entirely new economic system has been developed in this age of mechanical power, and in that sense the engineering of the present day is modern, and the relations of the engineer to the community are more important and intimate; but such is the force of precedent that these relations have undergone very little change in all past history.

There is now rapidly developing a wave of collectivism, so to speak, or class power, engendered by the ease of over-production and the education of the masses. It will render the community work of the engineer more easy and also more necessary, for, if individualism is not protected, we must drift toward the jungle.

In our democratic aim to give equality of opportunity, we are losing sight of the fact that progress and civilization itself rest upon the achievements of individuals, and this is being demonstrated clearly in the present war, as it is in every crisis.

Invention and initiative require special opportunities and encouragement, and efficiency demands that those who are to direct shall be

selected through the exercise of wisdom, and it must be admitted that the policies, practices, and tendencies of the present time are not adapted to achieve this result.

The operation of competition to select the most fit is being more and more restricted, both by law and through combinations of men, and no process is proposed by which the survival of the fittest may be assured.

Admitting that the efforts of any individual are directed to securing existence, comfort, and power for himself, he is best benefited by a system which shall produce for the community the greatest total wealth.

The engineer, by his education and technical training, and especially by the confidence of all classes of the community which he enjoys, is well prepared to enter actively into public discussions and public service, and should do so.

To that end the engineer should form local associations, devoted, beyond the consideration of the technical and scientific aspects of engineering, to the larger service to the community as a co-operative part in public affairs.

One great lesson of the war is that the community should always have men of experience qualifying them to act as leaders in the various lines of activity when the emergency of war arises, and masses of men whose training would permit of their being quickly formed into an army.

It would seem as if this might be accomplished without the necessity of a large standing army and compulsory military training, which would not only involve a loss of energy, but would be a constant menace to peace.

When we consider that the cost of our greatest public work, the Panama Canal, represents but ten days' current cost of the war to England, it is evident that the construction of public works containing an element of military preparedness is justified, especially such works as will have an economic value commensurate with their cost independent of the war element.

The class of men liable to military service might be given periodic labor experience sufficient to keep them in physical condition for the emergencies of war, without much sacrifice of their normal occupations.

Engineers could well afford to supplement their technical training with a period of labor, especially in skilled lines.

Such public works would form a reservoir of employment for surplus labor in times of industrial depression. They would include the development of water powers, the construction of impounding reservoirs for irrigation and navigation purposes, canals and improvements of rivers, the reclamation of waste lands, and the needed enlargement of our railways, terminals, highways, etc.

In spite of days of work and nights of study which is the lot of the engineer, time must still be devoted to broader interests, and there is great need of such service.

With the greatest heritage of natural wealth in all the world, we are most prodigal and wasteful in its use. We have the largest supply of coal of any country, and, knowing that civilization itself, in a modern sense, depends upon the coal supply, we are told that in the short space of 200 years, if its consumption increases at the present rate, our coal will be exhausted. What a brief period compared with the 5 000 years of recorded history of China, which, with a coal supply next in size to our own, imports coal. The conservation of this mineral devolves upon the engineer, not only in the improvement of machinery with which its power is applied, but in the utilization of water power, the improvement of natural waterways, and the construction of canals, by which transportation costs can be lessened.

Eighty years ago, or within the lives of many now living, the canals of the United States, more than 2 500 miles, exceeded the railways in length. It was then possible to make the trip by water, completely around the eastern part of the country, from New York to Buffalo, by river and canal, thence to Cleveland by lake, to the Ohio River by canal, and thence by river to the Gulf.

An English engineer, writing of New Orleans at that time, said: "At every hour, I might almost say, every minute of the day, magnificent steamboats which convey passengers into the heart of the western country, fire off their signal guns and dash away at a rate that makes me giddy to think of."

At that time there were boats on the Mississippi River with a capacity of more than 1 000 tons, which exceeds the capacity of any of the present boats; and, subsequently, boats with a capacity of more than 3 000 tons were operated.

Previous improvements of our rivers have been piecemealed; we have aimed at quick results instead of conforming to a comprehensive

scheme. Navigation has faded away in regions where millions have been spent to preserve it. Flood lands which, by proper regulation of the river and by the deposition of sediment, might have been built up, have been protected by expensive levees which have prevented the natural process of reclamation. Floods which should be impounded for service in dry times are allowed to run to waste.

We are taught that the mouth of the Mississippi was once at Cairo, more than 1 000 miles from the Gulf, by the river, and that the great fertile valley was formed by alluvial deposits. These alluvial deposits are not only needed to raise the surface of the land, but are very rich in fertility. We have leveed the river so that the sediment is deposited in the Gulf, where it is not only lost to land building but entails constant labor in jetty building and dredging to keep the mouth of the river open to navigation. The levees are being built higher and higher at great cost, and the lands they are designed to protect are being eroded and their fertility exhausted. This would seem to be a problem where the engineer might "direct" the forces of Nature for the "use and convenience of Man" instead of opposing them.

Man is the only animal that invents. He can thus change his environment, which in turn re-acts to change him. We rise from purely animal conditions by the aid of invention. We can sink back by its loss. Little do they realize, who claim that labor produces all wealth, how paltry would be the results of labor without invention and design.

The Constitution aims to encourage invention. It is a common thing to find organized opposition to it, as for instance the proscription of the use of patented articles in city work, and the efforts of construction, industrial, and railway organizations to fight patents, and the professional opposition to their exploitation.

Of course, one with an established business is naturally placed in opposition to a monopoly which threatens successful competition or possibly ruin to that business. Further, the inventor is usually unable to perfect his inventions through want of means. It would seem that a way should be found whereby the community would profit in encouraging invention probably by granting less monopoly to the inventor, but on the other hand, rendering him more help. Invention is an individual matter. Committees cannot invent.

Although invention is quite apart from engineering, the work of the engineer is largely based on former inventions.

In the past year our Society joined with the other Founder Societies in the formation of the Engineering Council, designed to represent the present growing membership of more than 30 000 engineers in the consideration of public matters.

Though in itself the greatest step that has been taken looking to the advancement of the Profession, the movement is especially inspiring as indicating the determination of the engineers to co-operate. At the magnificent celebration at which we were welcomed to our new home by the other Founder Societies, this sentiment was expressed by all the speakers in entire accord.

With such unanimity of purpose, our prospects are indeed encouraging.

Other steps are necessary, however: The Council must act with conservatism commensurate with its dignity and responsibility. It would seem that an organization should be formed devoted to the material interests of the younger men, draftsmen, instrument men, inspectors, etc., with such association with the older engineers that their development and progress may be encouraged; in brief, an alliance of all engineering interests which would be in harmony with similar organizations in nearly all lines of endeavor.

To co-operate effectively with other classes of the community, we must be organized for concerted action in our own sphere.

The prompt and intelligent direction that could have been given to the engineering forces in preparation for the war, had such an organization existed, would have been of great advantage, not to speak of the benefits to the engineers themselves.

The lawyer whose work is in direct contact with the people, and whose public services as law-maker as well as expounder enables him to protect his interests, has the further advantage of public prominence which often leads to his selection on commissions of a technical nature, to the exclusion of the engineer.

The members of the medical profession long ago recognized the necessity of impressing the people with the value of their public services. Schools of contrary theories have co-operated for a common end. Laws governing sanitation and hygiene have resulted, and frequent accounts of their work appear in the press. They have unselfishly disclosed what were formerly regarded as professional secrets, and have gained the well-merited recognition of the public.

The Engineer should profit by example.

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Paper No. 1405

SPECIFICATIONS AND METHODS OF TESTS FOR PORTLAND CEMENT

The following specifications and methods of tests for Portland cement are those recommended by the Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement. The Conference was created for the purpose of securing uniformity and reconciling differences in cement specifications and methods of tests. The membership of the Conference was constituted as follows:

AMERICAN SOCIETY OF CIVIL ENGINEERS

Alfred Noble*
George S. Webster
Richard L. Humphrey

AMERICAN SOCIETY FOR TESTING MATERIALS

George F. Swain
Olaf Hoff
Clifford Richardson

UNITED STATES GOVERNMENT

Arthur P. Davis
Asa E. Phillips
Rudolph J. Wig

The Conference was organized on October 24th, 1912. Many data relating to the subjects under consideration were gathered from manufacturers, users, laboratories, and individuals. In addition, many original data were secured from experiments and tests conducted by the Conference or under its direction. The results were correlated and studied, and these, together with the conclusions based upon them,

* Deceased.

were published by the Conference in reports dated April 28th, 1915, and June 1st, 1916.

In final form, as here presented, the specifications and methods of tests were transmitted to the Society on January 16th, 1917.

SPECIFICATIONS.

1.—Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion, an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum. Definition.

I.—Chemical Properties.

2.—The following limits shall not be exceeded: Chemical Limits.

Loss on ignition, per cent	4.00
Insoluble residue, per cent	0.85
Sulfuric anhydride (SO_3), per cent	2.00
Magnesia (MgO), per cent	5.00

II.—Physical Properties.

3.—The specific gravity of cement shall be not less than 3.10 (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific gravity test will not be made unless specifically ordered. Specific Gravity.

4.—The residue on a standard No. 200 sieve shall not exceed 22 per cent. by weight. Fineness.

5.—A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness. Soundness.

6.—The cement shall not develop initial set in less than 45 minutes when the Vicat needle is used or 60 minutes when the Gillmore needle is used. Final set shall be attained within 10 hours. Time of Setting.

7.—The average tensile strength in pounds per square inch of not less than three standard mortar briquettes (see Section 51) composed of one part cement and three parts standard sand, by weight, shall be equal to or higher than the following: Tensile Strength.

Age at test, in days.	Storage of briquettes.	Tensile strength, in pounds per square inch.
7	1 day in moist air, 6 days in water	200
28	1 day in moist air, 27 days in water	300

8.—The average tensile strength of standard mortar at 28 days shall be higher than the strength at 7 days.

III.—Packages, Marking, and Storage.

Packages and Marking. *9.*—The cement shall be delivered in suitable bags or barrels with the brand and name of the manufacturer plainly marked thereon, unless shipped in bulk. A bag shall contain 94 lb. net. A barrel shall contain 376 lb. net.

Storage. *10.*—The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness.

IV.—Inspection.

Inspection. *11.*—Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least 10 days from the time of sampling shall be allowed for the completion of the 7-day test, and at least 31 days shall be allowed for the completion of the 28-day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The 28-day test shall be waived only when specifically so ordered.

V.—Rejection.

Rejection. *12.*—The cement may be rejected if it fails to meet any of the requirements of these specifications.

13.—Cement shall not be rejected on account of failure to meet the fineness requirement if upon retest after drying at 100° C. for one hour it meets this requirement.

14.—Cement failing to meet the test for soundness in steam may be accepted if it passes a retest using a new sample at any time within 28 days thereafter.

15.—Packages varying more than 5 per cent. from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

TESTS.

VI.—Sampling.

Number of Samples. *16.*—Tests may be made on individual or composite samples as may be ordered. Each test sample should weigh at least 8 lb.

17.—(*a*) *Individual Sample.*—If sampled in cars one test sample shall be taken from each 50 bbl. or fraction thereof. If sampled in bins one sample shall be taken from each 100 bbl.

(*b*) *Composite Sample.*—If sampled in cars one sample shall be taken from one sack in each 40 sacks (or 1 bbl. in each 10 bbl.) and

combined to form one test sample. If sampled in bins or warehouses one test sample shall represent not more than 200 bbl.

18.—Cement may be sampled at the mill by any of the following methods that may be practicable, as ordered: Method of Sampling.

(*a*) *From the Conveyor Delivering to the Bin.*—At least 8 lb. of cement shall be taken from approximately each 100 bbl. passing over the conveyor.

(*b*) *From Filled Bins by Means of Proper Sampling Tubes.*—Tubes inserted vertically may be used for sampling cement to a maximum depth of 10 ft. Tubes inserted horizontally may be used where the construction of the bin permits. Samples shall be taken from points well distributed over the face of the bin.

(*c*) *From Filled Bins at Points of Discharge.*—Sufficient cement shall be drawn from the discharge openings to obtain samples representative of the cement contained in the bin, as determined by the appearance at the discharge openings of indicators placed on the surface of the cement directly above these openings before drawing of the cement is started.

19.—Samples preferably shall be shipped and stored in air-tight containers. Samples shall be passed through a sieve having 20 meshes per linear inch in order to thoroughly mix the sample, break up lumps and remove foreign materials. Treatment of Sample.

VII.—Chemical Analysis.

Loss on Ignition.

20.—One gram of cement shall be heated in a weighed covered platinum crucible, of 20 to 25-cc. capacity, as follows, using either method (*a*) or (*b*) as ordered: Method.

(*a*) The crucible shall be placed in a hole in an asbestos board, clamped horizontally so that about three-fifths of the crucible projects below, and blasted at a full red heat for 15 minutes with an inclined flame; the loss in weight shall be checked by a second blasting for 5 minutes. Care shall be taken to wipe off particles of asbestos that may adhere to the crucible when withdrawn from the hole in the board. Greater neatness and shortening of the time of heating are secured by making a hole to fit the crucible in a circular disk of sheet platinum and placing this disk over a somewhat larger hole in an asbestos board.

(*b*) The crucible shall be placed in a muffle at any temperature between 900 and 1 000° C. for 15 minutes, and the loss in weight shall be checked by a second heating for 5 minutes.

21.—A permissible variation of 0.25 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 4 per cent. Permissible Variation.

Insoluble Residue.

Method.

22.—To a 1-g. sample of cement shall be added 10 cc. of water and 5 cc. of concentrated hydrochloric acid; the liquid shall be warmed until effervescence ceases. The solution shall be diluted to 50 cc. and digested on a steam bath or hot plate until it is evident that decomposition of the cement is complete. The residue shall be filtered, washed with cold water, and the filter paper and contents digested in about 30 cc. of a 5-per cent. solution of sodium carbonate, the liquid being held at a temperature just short of boiling for 15 minutes. The remaining residue shall be filtered, washed with cold water, then with a few drops of hot hydrochloric acid, 1:9, and finally with hot water, and then ignited at a red heat and weighed as the insoluble residue.

Permissible Variation.

23.—A permissible variation of 0.15 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 0.85 per cent.

Sulfuric Anhydride.

Method.

24.—One gram of the cement shall be dissolved in 5 cc. of concentrated hydrochloric acid diluted with 5 cc. of water, with gentle warming; when solution is complete 40 cc. of water shall be added, the solution filtered, and the residue washed thoroughly with water. The solution shall be diluted to 250 cc., heated to boiling, and 10 cc. of a hot 10-per cent. solution of barium chloride shall be added slowly, drop by drop, from a pipette, and the boiling continued until the precipitate is well formed. The solution shall be digested on the steam bath until the precipitate has settled. The precipitate shall be filtered, washed, and the paper and contents placed in a weighed platinum crucible and the paper slowly charred and consumed without flaming. The barium sulfate shall then be ignited and weighed. The weight obtained multiplied by 34.3 gives the percentage of sulfuric anhydride. The acid filtrate obtained in the determination of the insoluble residue may be used for the estimation of sulfuric anhydride instead of using a separate sample.

Permissible Variation.

25.—A permissible variation of 0.10 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 2.00 per cent.

Magnesia.

Method.

26.—To 0.5 g. of the cement in an evaporating dish shall be added 10 cc. of water to prevent lumping and then 10 cc. of concentrated hydrochloric acid. The liquid shall be gently heated and agitated until attack is complete. The solution shall then be evaporated to complete dryness on a steam or water bath. To hasten dehydration the residue may be heated to 150 or even 200° C. for one-half to one hour. The residue shall be treated with 10 cc. of concentrated hydrochloric acid

diluted with an equal amount of water. The dish shall be covered and the solution digested for ten minutes on a steam bath or water bath. The diluted solution shall be filtered and the separated silica washed thoroughly with water.[1] Five cubic centimeters of concentrated hydrochloric acid and sufficient bromine water to precipitate any manganese which may be present, shall be added to the filtrate (about 250 cc.). This shall be made alkaline with ammonium hydroxide, boiled until there is but a faint odor of ammonia, and the precipitated iron and aluminum hydroxides, after settling, shall be washed with hot water, once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate shall be transferred by a jet of hot water to the precipitating vessel and dissolved in 10 cc. of hot hydrochloric acid. The paper shall be extracted with acid, the solution and washings being added to the main solution. The aluminum and iron shall then be reprecipitated at boiling heat by ammonium hydroxide and bromine water in a volume of about 100 cc., and the second precipitate shall be collected and washed on the filter used in the first instance if this is still intact. To the combined filtrates from the hydroxides of iron and aluminum, reduced in volume if need be, 1 cc. of ammonium hydroxide shall be added, the solution brought to boiling, 25 cc. of a saturated solution of boiling ammonium oxalate added, and the boiling continued until the precipitated calcium oxalate has assumed a well-defined granular form. The precipitate after one hour shall be filtered and washed, then with the filter shall be placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner; after ignition it shall be redissolved in hydrochloric acid and the solution diluted to 100 cc. Ammonia shall be added in slight excess, and the liquid boiled. The lime shall then be reprecipitated by ammonium oxalate, allowed to stand until settled, filtered and washed. The combined filtrates from the calcium precipitates shall be acidified with hydrochloric acid, concentrated on the steam bath to about 150 cc., and made slightly alkaline with ammonium hydroxide, boiled and filtered (to remove a little aluminum and iron and perhaps calcium). When cool, 10 cc. of saturated solution of sodium-ammonium-hydrogen phosphate shall be added with constant stirring. When the crystallin ammonium-magnesium orthophosphate has formed, ammonia shall be added in moderate excess. The solution shall be set aside for several hours in a cool place, filtered and washed with water containing 2.5 per cent. of NH_3. The precipitate shall be dissolved in a small quantity of hot hydrochloric acid, the solution diluted to about 100 cc., 1 cc. of a saturated solution of sodium-ammonium-hydrogen phosphate added, and ammonia drop by drop, with constant stirring, until the precipitate is again formed as described and the

[1] Since this procedure does not involve the determination of silica, a second evaporation is unnecessary.

ammonia is in moderate excess. The precipitate shall then be allowed to stand about two hours, filtered and washed as before. The paper and contents shall be placed in a weighed platinum crucible, the paper slowly charred, and the resulting carbon carefully burned off. The precipitate shall then be ignited to constant weight over a Meker burner, or a blast not strong enough to soften or melt the pyrophosphate. The weight of magnesium pyrophosphate obtained multiplied by 72.5 gives the percentage of magnesia. The precipitate so obtained always contains some calcium and usually small quantities of iron, aluminum, and manganese as phosphates.

Permissible Variation. *27.*—A permissible variation of 0.4 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported at 5.00 per cent.

VIII.—Determination of Specific Gravity.

Apparatus. *28.*—The determination of specific gravity shall be made with a standardized Le Chatelier apparatus which conforms to the requirements illustrated in Fig. 1. This apparatus is standardized by the United States Bureau of Standards. Kerosene free from water, or benzine not lighter than 62° Baumé, shall be used in making this determination.

Method. *29.*—The flask shall be filled with either of these liquids to a point on the stem between zero and one cubic centimeter, and 64 g. of cement, of the same temperature as the liquid, shall be slowly introduced, taking care that the cement does not adhere to the inside of the flask above the liquid and to free the cement from air by rolling the flask in an inclined position. After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; the difference between readings is the volume displaced by 64 g. of the cement.

The specific gravity shall then be obtained from the formula

$$\text{Specific gravity} = \frac{\text{Weight of cement (g.)}}{\text{Displaced volume (cc.)}}$$

30.—The flask, during the operation, shall be kept immersed in water, in order to avoid variations in the temperature of the liquid in the flask, which shall not exceed 0°.5 C. The results of repeated tests should agree within 0.01.

31.—The determination of specific gravity shall be made on the cement as received; if it falls below 3.10, a second determination shall be made after igniting the sample as described in Section 20.

IX.—Determination of Fineness.

Apparatus. *32.*—Wire cloth for standard sieves for cement shall be woven (not twilled) from brass, bronze, or other suitable wire, and mounted without distortion on frames not less than 1½ in. below the top of the frame.

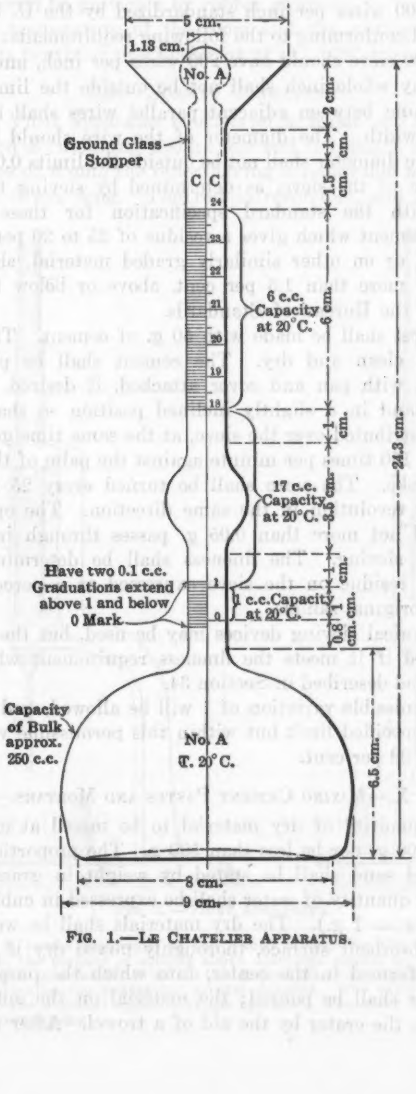


FIG. 1.—LE CHATELIER APPARATUS.

The sieve frames shall be circular, approximately 8 in. in diameter, and may be provided with a pan and cover.

33.—A standard No. 200 sieve is one having nominally a 0.0029-in. opening and 200 wires per inch standardized by the U. S. Bureau of Standards, and conforming to the following requirements:

The No. 200 sieve should have 200 wires per inch, and the number of wires in any whole inch shall not be outside the limits of 192 to 208. No opening between adjacent parallel wires shall be more than 0.0050 in. in width. The diameter of the wire should be 0.0021 in. and the average diameter shall not be outside the limits 0.0019 to 0.0023 in. The value of the sieve, as determined by sieving tests made in conformity with the standard specification for these tests on a standardized cement which gives a residue of 25 to 20 per cent. on the No. 200 sieve, or on other similarly graded material, shall not show a variation of more than 1.5 per cent. above or below the standards maintained at the Bureau of Standards.

Method. *34.*—The test shall be made with 50 g. of cement. The sieve shall be thoroughly clean and dry. The cement shall be placed on the No. 200 sieve, with pan and cover attached, if desired, and shall be held in one hand in a slightly inclined position so that the sample will be well distributed over the sieve, at the same time gently striking the side about 150 times per minute against the palm of the other hand on the up stroke. The sieve shall be turned every 25 strokes about one-sixth of a revolution in the same direction. The operation shall continue until not more than 0.05 g. passes through in one minute of continuous sieving. The fineness shall be determined from the weight of the residue on the sieve expressed as a percentage of the weight of the original sample.

35.—Mechanical sieving devices may be used, but the cement shall not be rejected if it meets the fineness requirement when tested by the hand method described in Section 34.

Permissible Variation. *36.*—A permissible variation of 1 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 22 per cent.

X.—Mixing Cement Pastes and Mortars.

Method. *37.*—The quantity of dry material to be mixed at one time shall not exceed 1 000 g. nor be less than 500 g. The proportions of cement or cement and sand shall be stated by weight in grams of the dry materials; the quantity of water shall be expressed in cubic centimeters (1 cc. of water = 1 g.). The dry materials shall be weighed, placed upon a non-absorbent surface, thoroughly mixed dry if sand is used, and a crater formed in the center, into which the proper percentage of clean water shall be poured; the material on the outer edge shall be turned into the crater by the aid of a trowel. After an interval of

½ minute for the absorption of the water the operation shall be completed by continuous, vigorous mixing, squeezing and kneading with the hands for at least one minute.[1] During the operation of mixing, the hands should be protected by rubber gloves.

38.—The temperature of the room and the mixing water shall be maintained as nearly as practicable at 21° C. (70° F.).

XI.—Normal Consistency.

39.—The Vicat apparatus consists of a frame *A* (Fig. 2) bearing a movable rod *B*, weighing 300 g., one end *C* being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle *D*, 1 mm. in diameter, 6 cm. long. The rod is reversible, and can be held in Apparatus.

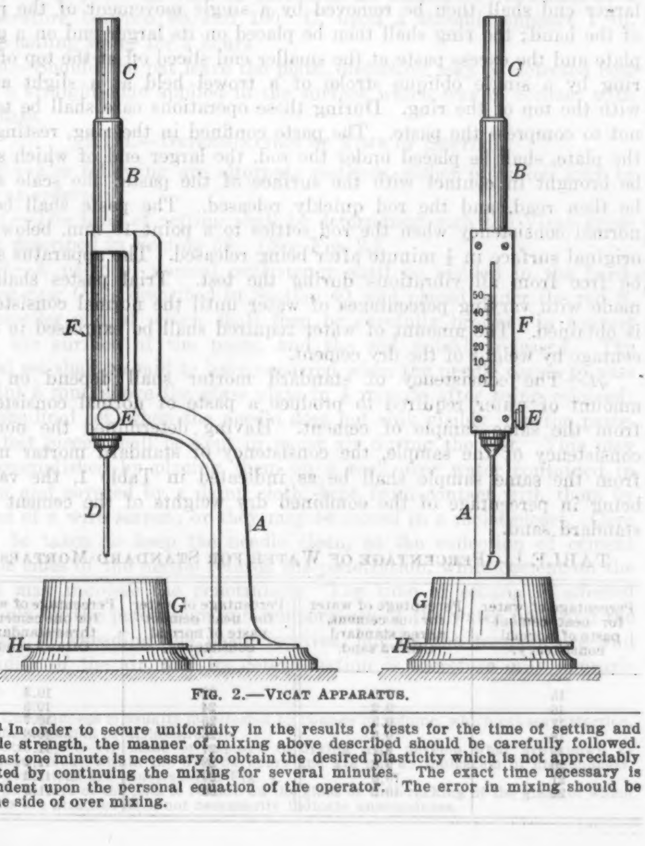


FIG. 2.—VICAT APPARATUS.

[1] In order to secure uniformity in the results of tests for the time of setting and tensile strength, the manner of mixing above described should be carefully followed. At least one minute is necessary to obtain the desired plasticity which is not appreciably affected by continuing the mixing for several minutes. The exact time necessary is dependent upon the personal equation of the operator. The error in mixing should be on the side of over mixing.

any desired position by a screw *E*, and has midway between the ends a mark *F* which moves under a scale (graduated to millimeters) attached to the frame *A*. The paste is held in a conical, hard-rubber ring *G*, 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate *H*, about 10 cm. square.

Method. *40.*—In making the determination, 500 g. of cement, with a measured quantity of water, shall be kneaded into a paste, as described in Section 37, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 in. apart; the ball resting in the palm of one hand shall be pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end shall then be removed by a single movement of the palm of the hand; the ring shall then be placed on its larger end on a glass plate and the excess paste at the smaller end sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care shall be taken not to compress the paste. The paste confined in the ring, resting on the plate, shall be placed under the rod, the larger end of which shall be brought in contact with the surface of the paste; the scale shall be then read, and the rod quickly released. The paste shall be of normal consistency when the rod settles to a point 10 mm. below the original surface in ½ minute after being released. The apparatus shall be free from all vibrations during the test. Trial pastes shall be made with varying percentages of water until the normal consistency is obtained. The amount of water required shall be expressed in percentage by weight of the dry cement.

41.—The consistency of standard mortar shall depend on the amount of water required to produce a paste of normal consistency from the same sample of cement. Having determined the normal consistency of the sample, the consistency of standard mortar made from the same sample shall be as indicated in Table 1, the values being in percentage of the combined dry weights of the cement and standard sand.

TABLE 1.—PERCENTAGE OF WATER FOR STANDARD MORTARS.

Percentage of water for neat cement paste of normal consistency.	Percentage of water for one cement, three standard Ottawa sand.	Percentage of water for neat cement paste of normal consistency.	Percentage of water for one cement, three standard Ottawa sand.
15	9.0	23	10.3
16	9.2	24	10.5
17	9.3	25	10.7
18	9.5	26	10.8
19	9.7	27	11.0
20	9.8	28	11.2
21	10.0	29	11.3
22	10.2	30	11.5

XII.—Determination of Soundness.[1]

42.—A steam apparatus, which can be maintained at a temperature between 98 and 100° C., or one similar to that shown in Fig. 3, is recommended. The capacity of this apparatus may be increased by using a rack for holding the pats in a vertical or inclined position. **Apparatus.**

43.—A pat from cement paste of normal consistency about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be made on clean glass plates about 4 in. square, and stored in moist air for 24 hours. In molding the pat, the cement paste shall first be flattened on the glass and the pat then formed by drawing the trowel from the outer edge toward the center. **Method.**

44.—The pat shall then be placed in an atmosphere of steam at a temperature between 98 and 100° C. upon a suitable support 1 in. above boiling water for 5 hours.

45.—Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

XIII.—Determination of Time of Setting.

46.—The following are alternate methods, either of which may be used as ordered:

47.—The time of setting shall be determined with the Vicat apparatus described in Section 39. (See Fig. 2.) **Vicat Apparatus.**

48.—A paste of normal consistency shall be molded in the hard-rubber ring *G*, as described in Section 40, and placed under the rod *B*, the smaller end of which shall then be carefully brought in contact with the surface of the paste, and the rod quickly released. The initial set shall be said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate in ½ minute after being released; and the final set, when the needle does not sink visibly into the paste. The test pieces shall be kept in moist air during the test. This may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth, kept from contact with them by means of a wire screen; or they may be stored in a moist closet. Care shall be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration. The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is therefore only approximate. **Vicat Method.**

[1] Unsoundness is usually manifested by change in volume, which causes distortion, cracking, checking or disintegration.

Pats improperly made or exposed to drying may develop what are known as shrinkage cracks within the first 24 hours and are not an indication of unsoundness. These conditions are illustrated in Fig. 4.

The failure of the pats to remain on the glass or the cracking of the glass to which the pats are attached does not necessarily indicate unsoundness.

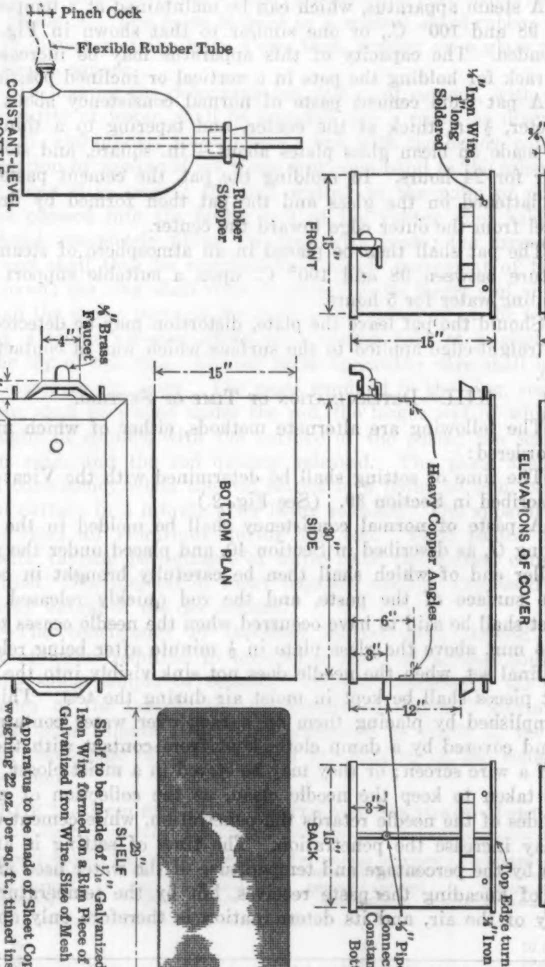


FIG. 3.—APPARATUS FOR MAKING SOUNDNESS TEST OF CEMENT.

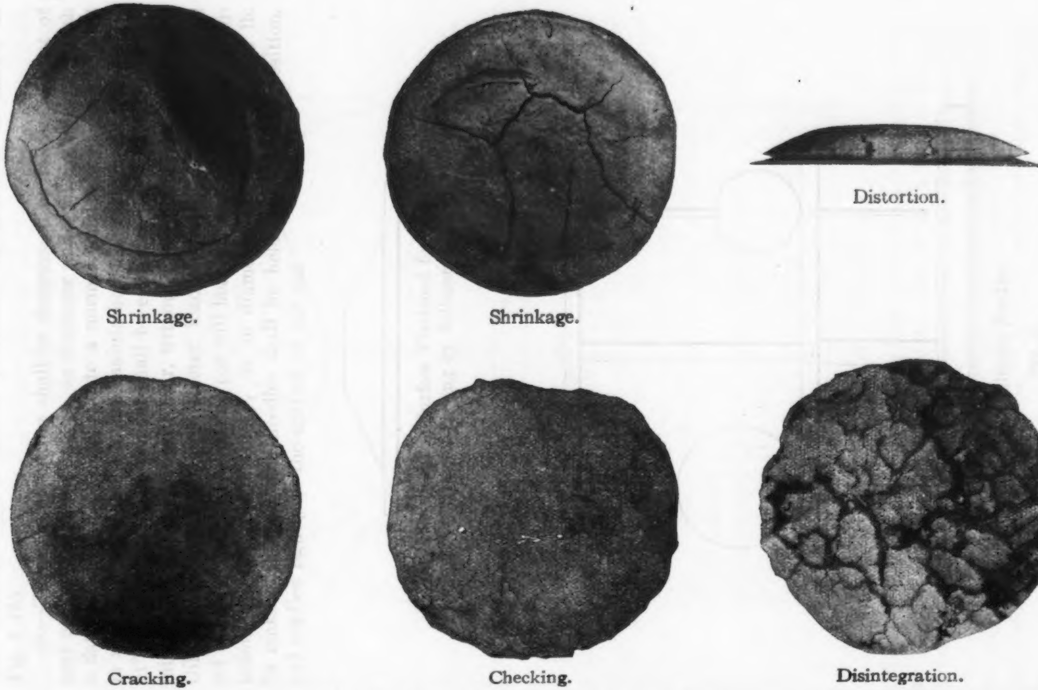
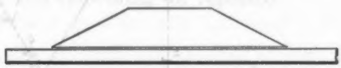


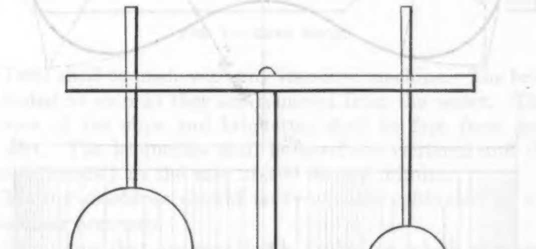
FIG. 4.—TYPICAL FAILURES IN SOUNDNESS TEST.

49.—The time of setting shall be determined by the Gillmore needles. The Gillmore needles should preferably be mounted as shown in Fig. 5 (*b*). **Gillmore Needles.**

50.—The time of setting shall be determined as follows: A pat of neat cement paste, about 3 in. in diameter and ½ in. in thickness with a flat top (Fig. 5 (*a*)), mixed to a normal consistency, shall be kept in moist air at a temperature maintained as nearly as practicable at 21° C. (70° F.). The cement shall be considered to have acquired its initial set when the pat will bear, without appreciable indentation, the Gillmore needle $\frac{1}{12}$ in. in diameter, loaded to weigh ¼ lb. The final set has been acquired when the pat will bear without appreciable indentation, the Gillmore needle $\frac{1}{24}$ in. in diameter, loaded to weigh 1 lb. In making the test, the needles shall be held in a vertical position, and applied lightly to the surface of the pat. **Gillmore Method.**



(*a*) Pat with Top Surface Flattened for Determining Time of Setting by Gillmore Method.



(*b*) Gillmore Needles.

FIG. 5.

XIV.—Tension Tests.

Form of Test Piece.

51.—The form of test piece shown in Fig. 6 shall be used. The molds shall be made of non-corroding metal and have sufficient material in the sides to prevent spreading during molding. Gang molds when used shall be of the type shown in Fig. 7. Molds shall be wiped with an oily cloth before using.

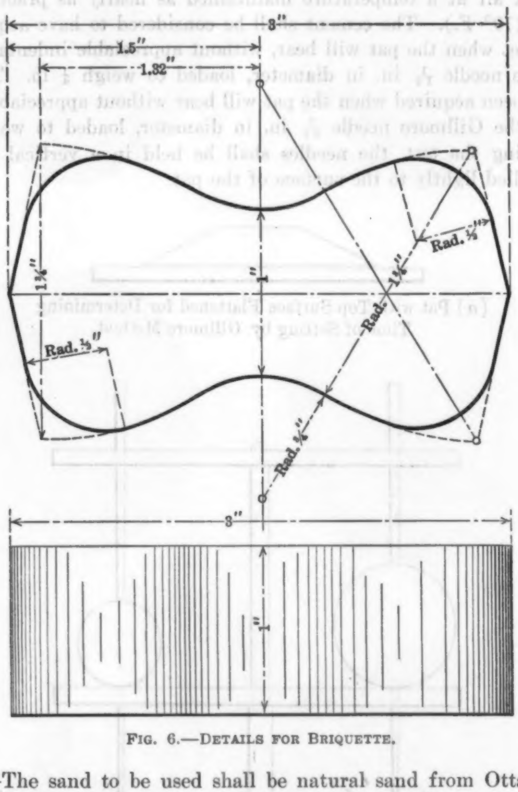


Fig. 6.—Details for Briquette.

Standard Sand.

52.—The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve. This sand may be obtained from the Ottawa Silica Co., at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill.

53.—This sand, having passed the No. 20 sieve, shall be considered standard when not more than 5 g. pass the No. 30 sieve after one minute continuous sieving of a 500-g. sample.

54.—The sieves shall conform to the following specifications:

The No. 20 sieve shall have between 19.5 and 20.5 wires per whole inch of the warp wires and between 19 and 21 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0165 in. and the average diameter shall not be outside the limits of 0.0160 and 0.0170 in.

The No. 30 sieve shall have between 29.5 and 30.5 wires per whole inch of the warp wires and between 28.5 and 31.5 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0110 in. and the average diameter shall not be outside the limits 0.0105 to 0.0115 in.

55.—Immediately after mixing, the standard mortar shall be placed in the molds, pressed in firmly with the thumbs and smoothed off with a trowel without ramming. Additional mortar shall be heaped above the mold and smoothed off with a trowel; the trowel shall be drawn over the mold in such a manner as to exert a moderate pressure on the material. The mold shall then be turned over and the operation of heaping, thumbing and smoothing off repeated. Molding.

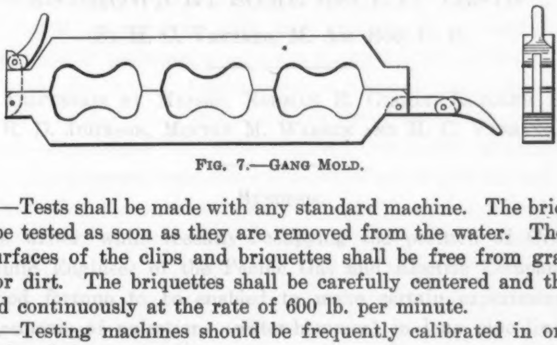


FIG. 7.—GANG MOLD.

56.—Tests shall be made with any standard machine. The briquettes shall be tested as soon as they are removed from the water. The bearing surfaces of the clips and briquettes shall be free from grains of sand or dirt. The briquettes shall be carefully centered and the load applied continuously at the rate of 600 lb. per minute. Testing.

57.—Testing machines should be frequently calibrated in order to determine their accuracy.

58.—Briquettes that are manifestly faulty, or which give strengths differing more than 15 per cent. from the average value of all test pieces made from the same sample and broken at the same period, shall not be considered in determining the tensile strength. Faulty Briquettes.

XV.—STORAGE OF TEST PIECES.

59.—The moist closet may consist of a soapstone, slate or concrete box, or a wooden box lined with metal. If a wooden box is used, the interior should be covered with felt or broad wicking kept wet. The bottom of the moist closet should be covered with water. The interior of the closet should be provided with non-absorbent shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily. Apparatus.

Methods.

60.—Unless otherwise specified, all test pieces, immediately after molding, shall be placed in the moist closet for from 20 to 24 hours.

61.—The briquettes shall be kept in molds on glass plates in the moist closet for at least 20 hours. After 24 hours in moist air the briquettes shall be immersed in clean water in storage tanks of non-corroding material.

62.—The air and water shall be maintained as nearly as practicable at a temperature of 21° C. (70° F.).

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Paper No. 1406

PULSATIONS IN PIPE LINES, AS SHOWN BY SOME RECENT TESTS*

By H. C. Vensano, M. Am. Soc. C. E.

With Discussion by Messrs. Norman R. Gibson, Rudolph Hering, R. D. Johnson, Minton M. Warren and H. C. Vensano.

Synopsis.

The writer, while recently occupying the position of Civil and Hydraulic Engineer of the Pacific Gas and Electric Company, had the good fortune to be enabled to make certain experiments and measurements of pulsations (water-hammer) in long pipe lines, and these are described in this paper. This information, it is believed, will prove a valuable addition to the present literature on the subject. The experiments should be of particular value because they were made on a line under actual operating conditions. They show what can be expected, practically, in the way of wave effects, and demonstrate that pulsations, whether due to gate opening or closing, can by no means be neglected in design, even for lines which are controlled by slowly moving gates.

The Drum Power Plant, of the Pacific Gas and Electric Company, put into service in 1913, is a hydro-electric generating station on the Bear River, in Placer County, California, in which there are

* Presented at the meeting of November 7th, 1917.

two 12 500-kw. electric generators, each driven by a pair of Pelton impulse wheels. The supply for these wheels is brought to the power-house in a riveted steel pipe, or penstock, 6 282 ft. long, and having a diameter of 72 in. at its upper end and 52 in. at the power-house. The experiments discussed herein were made on this penstock. Pulsations of acceleration due to gate opening and also of retardation due to gate closing were investigated. These effects were obtained by opening or closing (simultaneously) one or more of the needle nozzles which control the supply to the wheels. Owing to the fact that the time of closure for a nozzle was approximately 69 sec., the results will apply to the conditions of slow gate closure. Inasmuch as most of the experiments in the past (as far as the writer knows) have been on rapid gate closures (so rapid as to produce as nearly as possible an instantaneous condition), and inasmuch as the conditions usually occurring in practice deal with slow closure, the following results should be of considerable value in assisting the engineer to ascertain the water-hammer to be expected in penstock lines under practical conditions of design and operation.

A study of the curves obtained from the experiments described herein has led the writer to the following conclusions:

First.—That the general formula for pressure variation from normal at any point in a pipe line, with uniformly varying gate opening, should be

$$h = \frac{2\,(L_1 V_1 + L_2 V_2 + \;.\;.\;.\;.\; L_{x-1} V_{x-1} + L_x V_x)}{g\,T} \ldots (1)$$

This formula applies to a pipe with varying diameter.

Second.—For slow gate closure, this formula reduces to

$$h = \frac{2\,(L_1 V_1 + L_2 V_2 + L_3 V_3 +, \text{ etc. for the full length of the pipe})}{g\,T} \ldots (2)$$

and, further, reduces to

$$h = \frac{2\,L_1\,V_1}{g\,T} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

for slow gate closure with a pipe of uniform diameter, as advocated by Professor Joukovsky. Formulas (1), (2), and (3) are limited to a maximum value of

$$h = \frac{a_1\,v_1}{g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Third.—That, as this formula indicates, the velocity of flow at the gate, or at the point where the pressure is to be ascertained, does not necessarily (of itself alone) fix the magnitude of the pressure wave at such point, but that the magnitude of the pressure wave is influenced by the varying velocities of the moving water column in all portions of the line between the point at which the pressure is to be ascertained and the reservoir.

Fourth.—That, under ordinary conditions, the water-hammer effect may, and does, produce as great a fall in pressure below the normal as it produces a rise above normal after the gate has been closed completely. In other words, the pressure vibrates back and forth above and below normal after gate closure.

Fifth.—That, in pipes of uniform diameter, the magnitude of pressure variation along the pipe line will vary directly as the time required for the wave to travel from any point in question to the reservoir and return to the same point, as advanced by Professor Joukovsky, provided the time of gate closure is greater than the half period of the pipe.

Sixth.—That the effect of accelerating the water column by gate opening is analogous to the effect of retardation in gate closing, except that the pressure variations have the opposite sign. The period of pulsation is the same. The chief difference is that the wave effects die out much more rapidly with opening than with closing, and this seems also to damp the vibration so rapidly that the full magnitude is obtained only for a short time.

Seventh.—That the synthetic method, used by Professor Joukovsky and Miss Simin to determine wave forms, can be used to good advantage in the study of such effects, and can be made to predict probable wave forms and magnitudes, if properly interpreted.

Eighth.—It is here found that the velocity of wave propagation in water, for riveted steel pipe, can be calculated approximately by the recognized formula:

$$a = \frac{12}{\sqrt{\frac{W}{g}\left(\frac{1}{K} + \frac{D}{E\,b}\right)}} \text{.....................} (5)$$

if proper allowance is made for the effect of joint details.

General Description of Experiments.

In order to avoid making this paper too long, the writer assumes that the reader is familiar with certain literature on the subject already available. In fact, before examining this paper, he should read at least two papers to which the writer refers, particularly "Water-Hammer", by Miss O. Simin*, and also that part of a paper entitled "Penstock and Surge-Tank Problems",† by Minton M. Warren, Assoc. M. Am. Soc. C. E., applying to pressure due to slow or ordinary gate closures, together with (and especially) the discussion on that paper by the writer. Although it will be found that the writer (due to the results of the experiments described herein) has modified in some particulars the views he expressed in that discussion, certain fundamental ideas were brought out there at considerable length, and will not be repeated herein.

The experiments were made on the penstock of the Drum Power-House, Fig. 1. The gate arrangement at the lower end of the pipe line consists of four needle nozzles controlling an equal number of Pelton impulse wheels. The arrangement of the nozzles is shown in Fig. 5. The nozzles are operated electrically, and tests were first made to determine the rate, period, and characteristics of their opening and closing. They were specially designed to give a uniform rate of gate opening, but practically do not quite do so. The opening and closing curves obtained are shown by Fig. 6, which gives the relation of area of opening to time of gate motion. The opening curve shows an almost perfectly uniform rate of opening. The closing curve shows a somewhat slow start, probably due to lag in the motor operating the gate, and a somewhat accelerated finish, perhaps due to the increasing water pressure behind the needle of the nozzle as the flow was checked. Each experimental point on the diagram is the result of a number of tests.

During the tests the nozzles—which are deflecting nozzles—were directed away from the wheels. It is well to note here that the arrangement is such that, for the full open position of either one or more nozzles, the velocity in the 26-in. pipe, to which Gauge No. 1 was attached, remains practically constant.

* *Proceedings*, American Water Works Assoc., 1904, p. 341. This is a review and elaboration of the article by Professor J. Joukovsky in *Memoirs*, Imperial Academy of Sciences, St. Petersburg, 1897, Vol. IX.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 238.

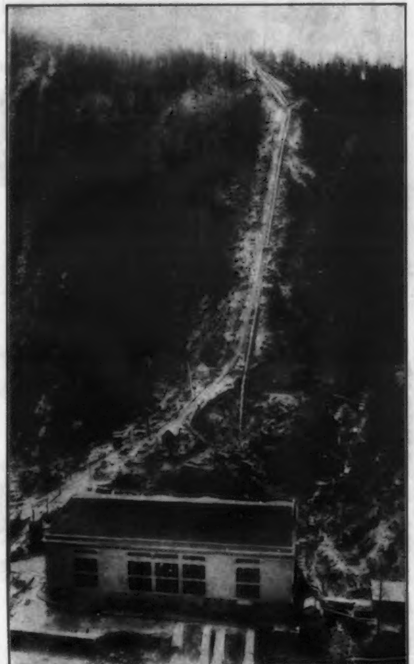


FIG. 1.—DRUM POWER-HOUSE, AND LOWER END OF PENSTOCK.

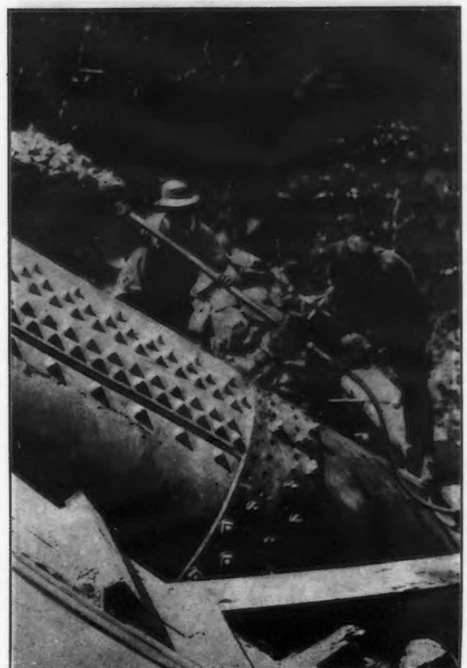


FIG. 2.—DETAIL OF PENSTOCK, DURING CONSTRUCTION.

FIG. 3.—DRUM PENSTOCK, 72 INCHES IN DIAMETER.

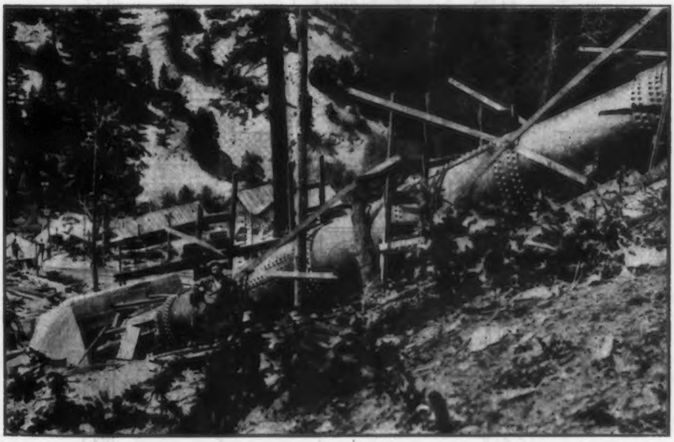
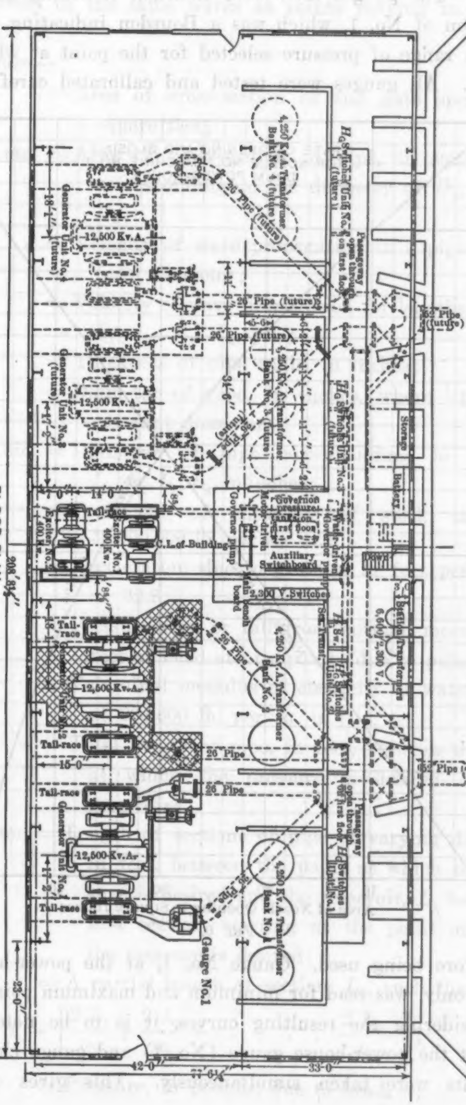


FIG. 4.—DRUM PENSTOCK, 72 INCHES IN DIAMETER.

Gauges.—The gauges used were Bourdon recording gauges with the exception of No. 1, which was a Bourdon indicating gauge, and each had a range of pressure selected for the point at which it was to be used. All gauges were tested and calibrated carefully imme-

GENERAL ARRANGEMENT OF DRUM POWER HOUSE



FLOOR PLAN

FIG. 5.

diately before being used. Gauge No. 1, at the power-house, being indicating only, was read for minimum and maximum points.

In considering the resulting curves, it is to be noted that the readings at the power-house gauge (No. 1) and pressure charts at two other points were taken simultaneously. This gives comparative

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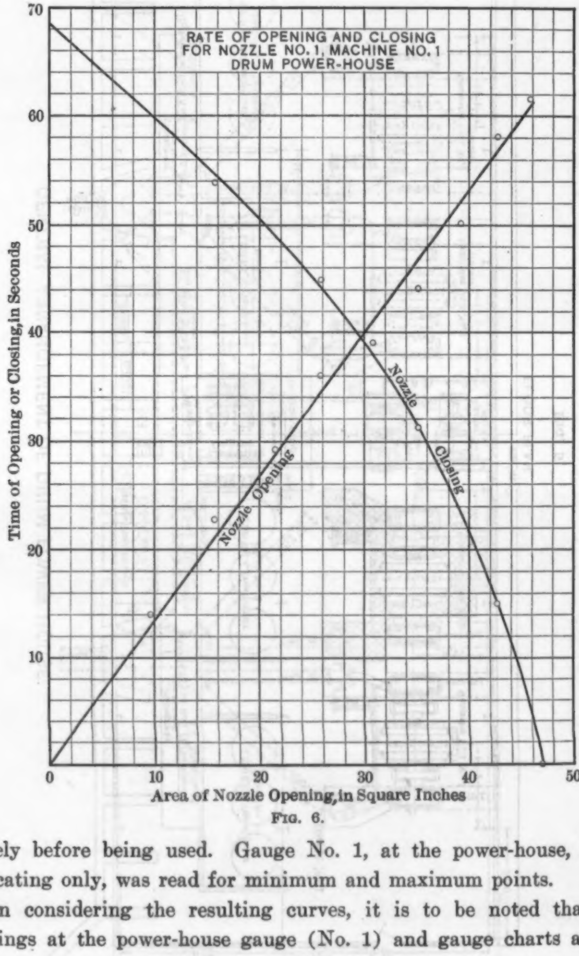


FIG. 6.

diately before being used. Gauge No. 1, at the power-house, being indicating only, was read for minimum and maximum points.

In considering the resulting curves, it is to be noted that the readings at the power-house gauge (No. 1) and gauge charts at two other points were taken simultaneously. This gives comparative

results for effects of the same waves at points varying in distance from the gate.

Nomenclature.—

A = Area of cross-section of full gate opening, in square feet;

A_1, A_2, A_3, A_4, etc. = Area of cross-section of pipe, in square feet, corresponding to the diameter of D_1, D_2, D_3, D_4;

a = Velocity of wave propagation in a pipe of uniform diameter;

a_1, a_2, a_3, a_4, = Velocity of wave propagation for lengths of pipe, L_1, L_2, L_3, L_4, etc.;

b = Thickness of pipe walls, in inches;

D = Diameter of pipe, in inches, where line is of constant diameter;

D_1, D_2, D_3, D_4, etc. = Diameter of pipe corresponding to lengths, L_1, L_2, L_3, L_4, in inches;

E = Coefficient of elasticity of steel, taken at 30 000 000 lb. per sq. in.;

g = Acceleration due to gravity, in feet per second = 32.2;

h = Pressure rise, or fall from normal, measured in feet of head at any point (due to pulsations);

k = Voluminal modulus of elasticity of water, taken at 294 000 lb. per sq. in.;

L = Total length of pipe, in feet, between the point at which the pressure is desired and the reservoir;

L_1, L_2, L_3, L_4, etc. = Length of sections of pipe of varying diameter, in feet, between the point at which the pressure is desired and the reservoir, L_1 being the first section, starting at the point at which the pressure is desired;

l_x = A partial length of section, L_x, such that

$$\frac{2\,l_x}{a_x} = T - \left[\frac{2\,L_1}{a_1} + \frac{2\,L_2}{a_2} + \quad . \quad . \quad . \quad . \quad \frac{2\,L_{x-1}}{a_{x-1}}\right];$$

p = Pressure, in pounds, due to head, h;

T = Time of gate closure (or opening), in seconds;

t = Time, in seconds, for wave to travel from any point to reservoir and return to that point;

t_0 = One-half period of pipe $= \frac{2L}{a}$ for pipe of uniform diameter, $= \frac{2L_1}{a_1} + \frac{2L_2}{a_2} + \frac{2L_3}{a_3}$, etc., for pipe of varying diameter, taking L_1 from the gate and including all sections to the reservoir;

V = Velocity of flow in pipe of uniform diameter, in feet per second, at beginning of gate motion;

V_1, V_2, V_3, V_4, etc. = Velocity of flow, in feet per second, in sections of pipe of diameter, D_1, D_2, D_3, D_4, respectively, at beginning of gate motion;

v_1, v_2, v_3, v_4, etc. = Velocity of flow, in feet per second, in sections of pipe of diameter, D_1, D_2, D_3, D_4, respectively, at end of gate motion;

w = Weight of water per cubic foot, taken at 62.4 lb.;

V_x, a_x, A_x, are values corresponding to V_1, a_1, A_1, at the point, X, in the pipe line.

Slow gate closure is defined as one in a time greater than t_0. Rapid gate closure is one in a time less than t_0.

Experiments.—The following experiments for various gate motions were made:

Test No. 1. July 15th, 1915.

Four nozzles opened simultaneously, left open 5 min., and closed simultaneously. Record taken at Gauge Points Nos. 2 and 3.

Test No. 2. July 15th, 1915.

Four nozzles opened simultaneously, left open 5 min., and closed simultaneously. Records taken at Gauge Points Nos. 2 and 3.

Test No. 3. July 15th, 1915.

Four nozzles opened simultaneously, left open 5 min., and closed simultaneously.

Chart No. 3-A (Fig. 7) Gauge No. 2. Both opening and closing.

Chart No. 3-B (Fig. 8) Gauge No. 3. Both opening and closing.

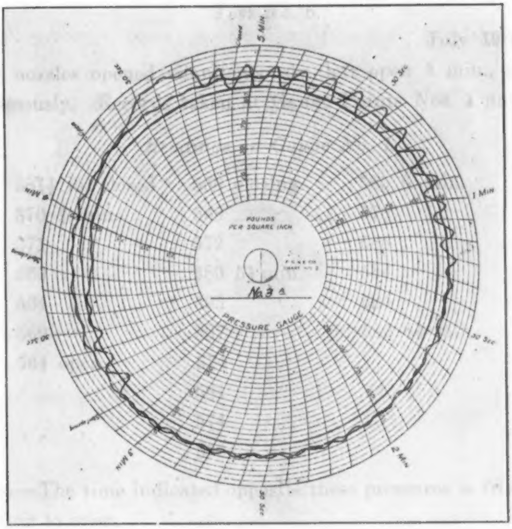

5 Min
1 Min
30 Sec
2 Min
POUNDS
PER SQUARE INCH
No. 3 A
PRESSURE GAUGE

FIG. 7.

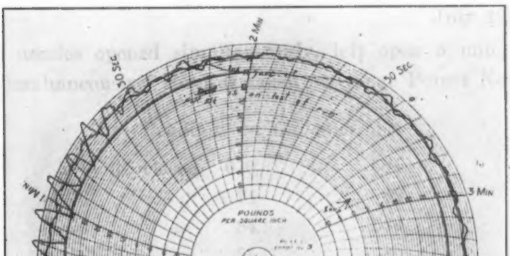

2 Min
50 Sec
3 Min
1 Min
30 Sec
4 Min
POUNDS
PER SQUARE INCH
No. 3 B
PRESSURE GAUGE

FIG. 8.

Test No. 6.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and closed simultaneously. Records taken at Gauge Points Nos. 1 and 2.

Readings on Gauge No. 1.

581½ lb. closed	566 closing	585 6 min.
570 ¾ min.	568	578
571	572	585
560	580 5½ min.	584
564	595	584
562	597	580 8½ min.
564 open	574	
	584	
	574	
	590	

Note.—The time indicated opposite these pressures is from the time of starting to open.

Test No. 7.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1 and 2.

Readings on Gauge No. 1.

581½ lb. closed	670	684
76	74	82
70	84	584
58	95	80
62	97	83
63	85 closed	80
64 open	78	83
665 closing	79	

Test No. 8.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1 and 2.

Readings on Gauge No. 1.

581½ lb.	closed	566	closing	574	584	582	
74		68		82	80	81	
72		70		88	84	81	7¼ min. total
62		74		78	83		
58		90		86	84		
64	open	98		78	83		
		96	closed	84	80		

Test No. 9.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and closed simultaneously.

Chart No. 9-B (Fig. 9) Gauge No. 2. Closing nozzles.
Chart No. 9-D (Fig. 10) Gauge No. 3. Closing nozzles.

Readings on Gauge No. 1.

581½ lb.	closed	568	closing	578	582	580	
74		72		86	80	84	8¼ min. total
72		80		82	82		
66		96		79	78		
58		96		86	84		
62		80	closed	80	80		
64	open	88		78	84		

Test No. 10.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and then closed simultaneously.

Chart No. 10-A (Fig. 11) Gauge No. 2. Opening nozzles.
Chart No. 10-C (Fig. 12) Gauge No. 3. Opening nozzles.

Readings on Gauge No. 1.

581½ lb.	closed	567		569	594		583	
72		558		72	96	closed	81	
76		564	open	80	80		82	
71		566	closing	98	84		82	Total time 7 min.

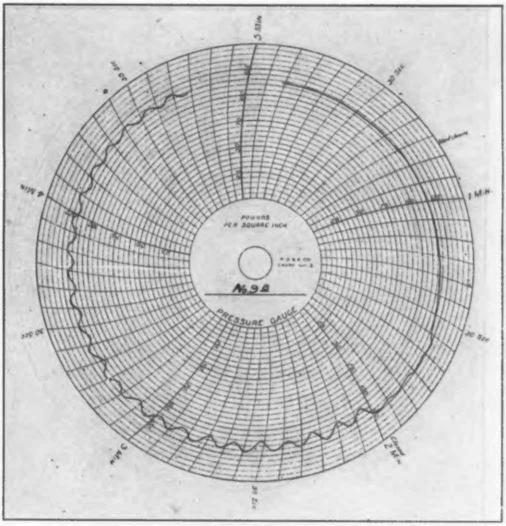

5 Min.
30 Sec.
1 Min.
30 Sec.
2 Min.
30 Sec.
3 Min.
30 Sec.
4 Min.
POUNDS
PER SQUARE INCH
No. 9 B
PRESSURE GAUGE

FIG. 9.

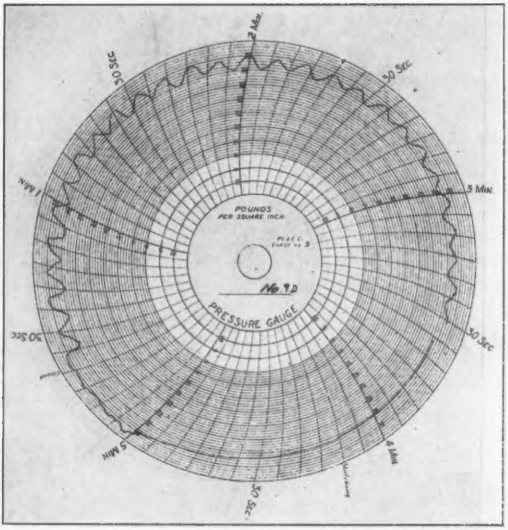

2 Min.
30 Sec.
3 Min.
30 Sec.
4 Min.
30 Sec.
5 Min.
30 Sec.
1 Min.
30 Sec.
POUNDS
PER SQUARE INCH
No. 9 D
PRESSURE GAUGE

FIG. 10.

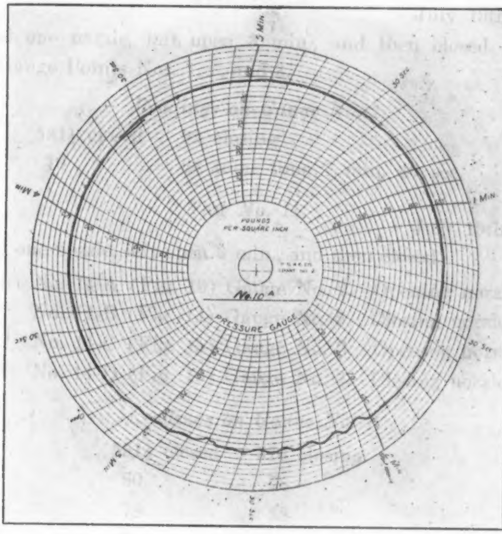

5 MIN
1 MIN
4 MIN
POUNDS
PER SQUARE INCH
No. 10A
PRESSURE GAUGE

FIG. 11.

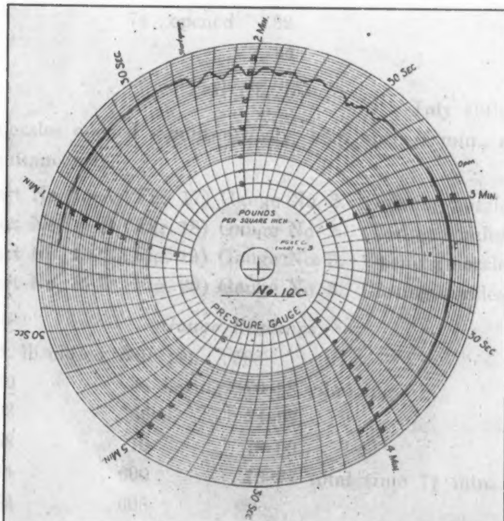

2 MIN
30 SEC
1 MIN
5 MIN.
POUNDS
PER SQUARE INCH
No. 10C
PRESSURE GAUGE
4 MIN.
Open

FIG. 12.

[illegible]

[illegible]

Test No. 13.

July 19th, 1915.

Opened one nozzle, left open 3 min., and then closed. Records taken at Gauge Points Nos. 1, 2, and 3.

Readings on Gauge No. 1.

581½ closed	92 closing
70	84 total time 7 min. 40 sec.

Test No. 14.

July 19th, 1915.

Opened one nozzle, left open 3 min., and then closed.

Chart No. 14-A (Fig. 13) Gauge No. 2. Opening nozzle.
" No. 14-B (Fig. 14) Gauge No. 2. Closing nozzle.
" No. 14-C (Fig. 15) Gauge No. 3. Opening nozzle.
" No. 14-D (Fig. 16) Gauge No. 3. Closing nozzle.

Readings on Gauge No. 1.

581½ closed	576 closing
80	78
78	88
70	92
72	90
74 opened	82
	84

Test No. 15.

July 19th, 1915.

Four nozzles opened simultaneously, held open 3 min., and then closed simultaneously.

Chart No. 15-A (Fig. 17) Gauge No. 2. Opening nozzles.
Chart No. 15-B (Fig. 18) Gauge No. 2. Closing nozzles.
Chart No. 15-C (Fig. 19) Gauge No. 3. Opening nozzles.
Chart No. 15-D (Fig. 20) Gauge No. 3. Closing nozzles.

Gauge Readings.

582 lb. closed	540 open	598
80	48 closing	570–98 vibrating
63	56	74–92 "
58	70	78–90 "
44	600	76–92 total time 7¾ min.
32	608	

Test No. 16.

July 19th, 1915.

Four nozzles opened simultaneously, held open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1, 2, and 3.

Chart No. 16-A (Fig. 21) Gauge No. 2. Opening nozzles.
Chart No. 16-B (Fig. 22) Gauge No. 2. Closing nozzles.

Readings on Gauge No. 1.

582½ lb. closed	534	556	610
70	38	70	575
62	40 open	608	80
58	48 closing	612	85 total time 7 min.

Test No. 18.

July 18th, 1915.

Opened two nozzles simultaneously, held open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1, 3, and 4.

Readings on Gauge No. 1.

582 closed	75	562	578	595
72	70	65	80	98 closed
78	70	66 open	84	
74	62	70 closing	88	Readings taken
80	61	72	97	about every ½
72	65	74	98	min.
76	65	76	96	

Note.—Evidently had a different man reading this gauge than for the previous experiments.

Test No. 19.

July 18th, 1915.

Opened two nozzles simultaneously, held open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1, 3, and 4.

Chart No. 19-A (Fig. 23) Gauge No. 4. Opening nozzles.
Chart No. 19-B (Fig. 24) Gauge No. 4. Closing nozzles.

Readings on Gauge No. 1.

582 closed	573 closing
570	599
560	582
566 open	583 closed. Readings taken every ½ min.

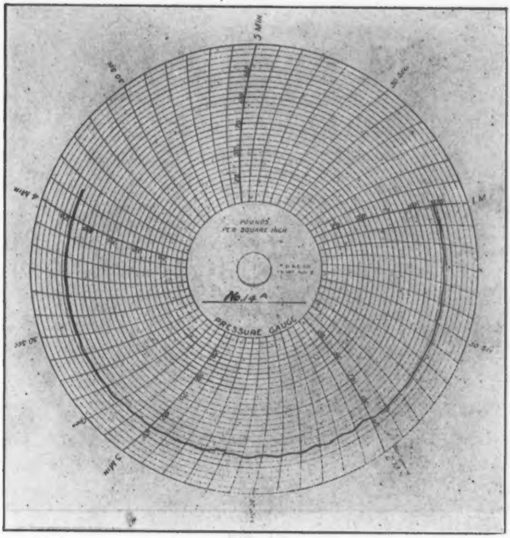

3 MIN
1 M
POUNDS
PER SQUARE INCH
No.14 A
PRESSURE GAUGE
4 Min
3 Min

FIG. 13.

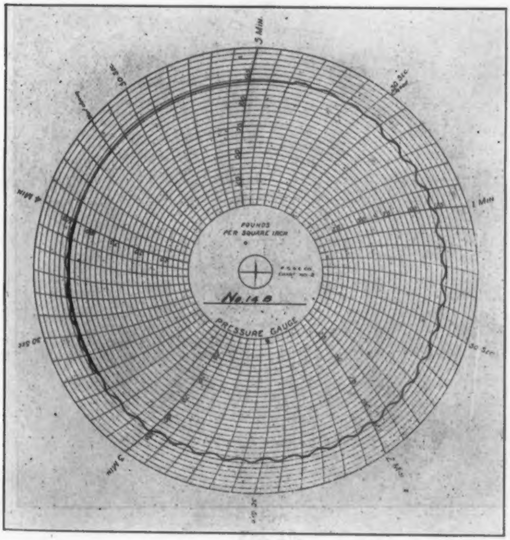

3 Min
1 Min
POUNDS
PER SQUARE INCH
No.14 B
PRESSURE GAUGE
4 Min
3 Min
2 Min

FIG. 14.

FIG. 13.

FIG. 14

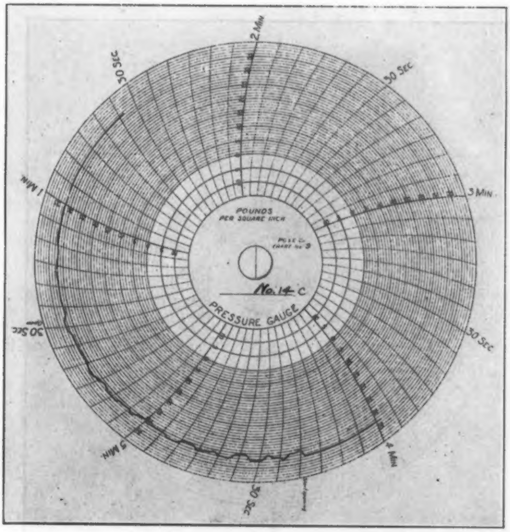

2 Min
30 Sec
30 Sec
1 Min
3 Min
POUNDS
PER SQUARE INCH
No. 14 C
PRESSURE GAUGE
30 Sec
30 Sec
5 Min
4 Min
30 Sec

FIG. 15.

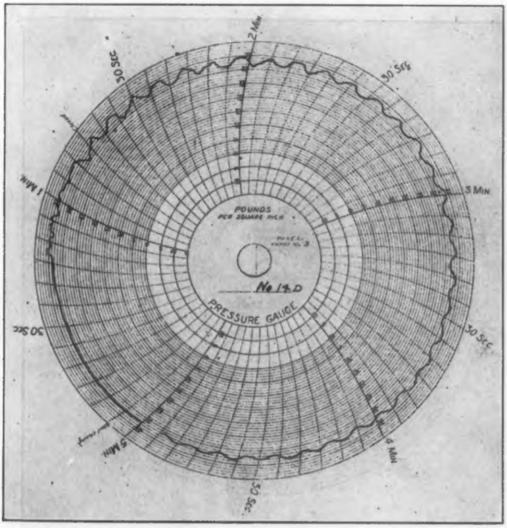
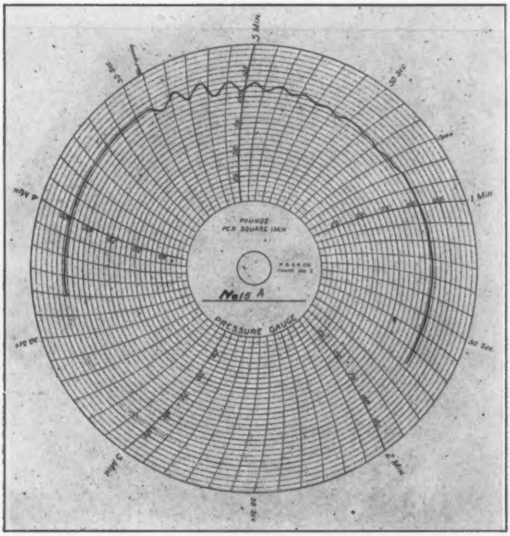
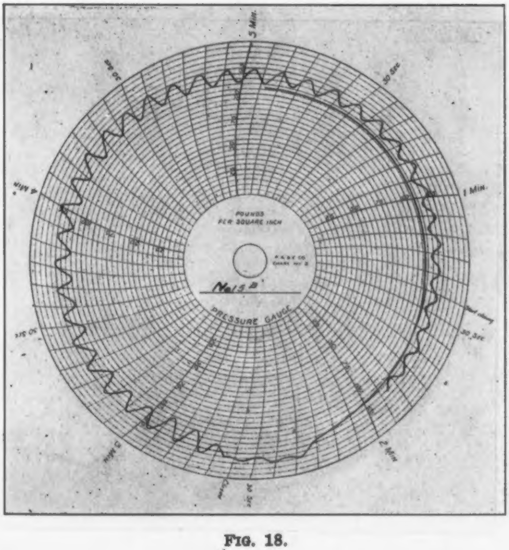

2 Min
30 Sec
30 Sec
1 Min
3 Min
POUNDS
PER SQUARE INCH
No. 14 D
PRESSURE GAUGE
30 Sec
30 Sec
5 Min
4 Min
30 Sec

FIG. 16.



POUNDS
PER SQUARE INCH
No15 A
PRESSURE GAUGE

Fig. 17.



POUNDS
PER SQUARE INCH
No15 B
PRESSURE GAUGE

Fig. 18.

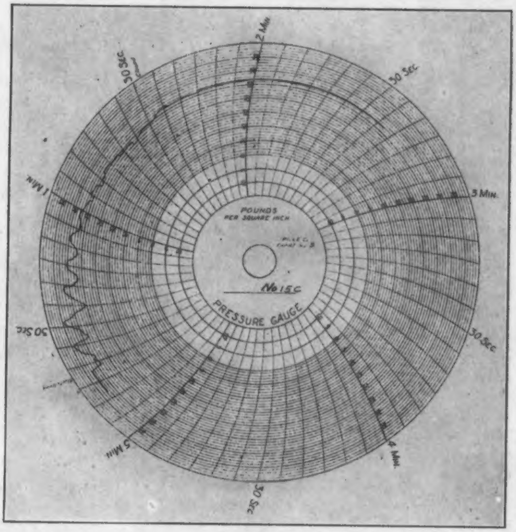

2 Min.
30 Sec.
30 Sec.
1 Min.
3 Min.
POUNDS
PER SQUARE INCH
No. 15C
PRESSURE GAUGE
30 Sec.
30 Sec.
5 Min.
4 Min.
30 Sec.

Fig. 19.

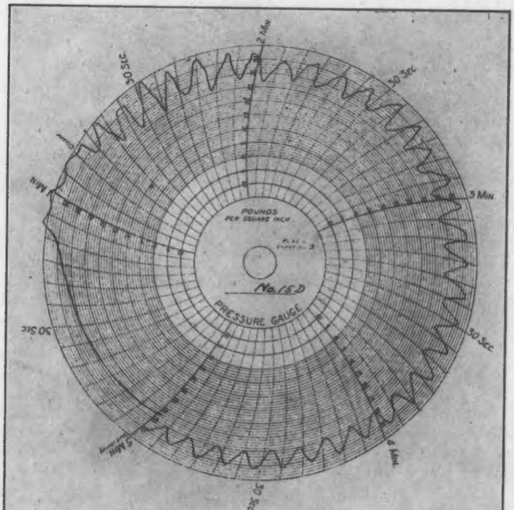

2 Min.
30 Sec.
30 Sec.
1 Min.
3 Min.
POUNDS
PER SQUARE INCH
No. 15D
PRESSURE GAUGE
30 Sec.
30 Sec.
5 Min.
4 Min.
30 Sec.

Fig. 20.

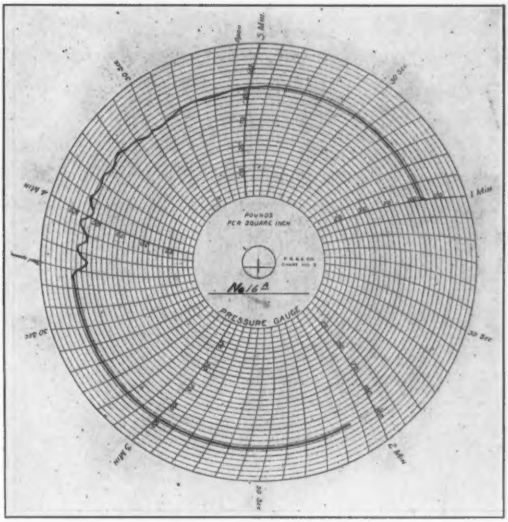

POUNDS
PER SQUARE INCH
No 16 A
PRESSURE GAUGE
1 Min
30 Sec
2 Min

FIG. 21.

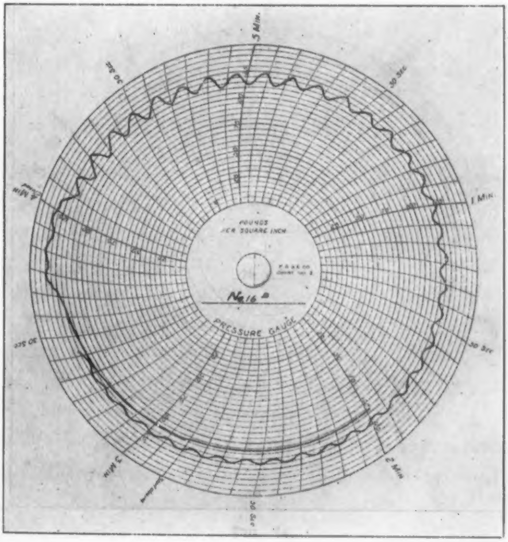

POUNDS
PER SQUARE INCH
No 16 B
PRESSURE GAUGE
1 Min
30 Sec
2 Min

FIG. 22.

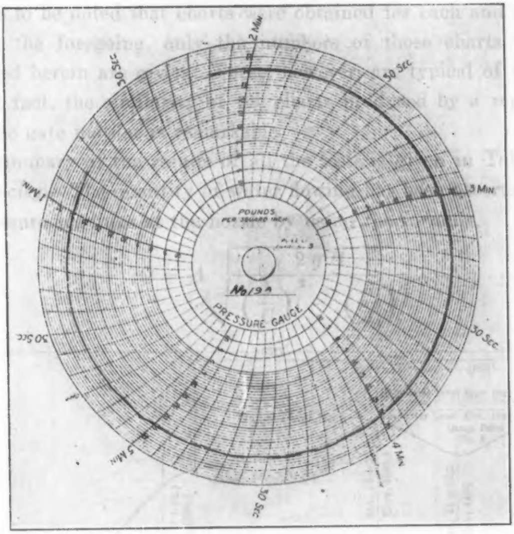

2 MIN
30 SEC
3 MIN
30 SEC
4 MIN
30 SEC
5 MIN
30 SEC
1 MIN
30 SEC
POUNDS
PER SQUARE INCH
No 19 A
PRESSURE GAUGE

FIG. 23.

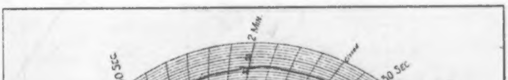

2 MIN
30 SEC
3 MIN
30 SEC
4 MIN
30 SEC
5 MIN
30 SEC
1 MIN
30 SEC
POUNDS
PER SQUARE INCH
No 19 B
PRESSURE GAUGE

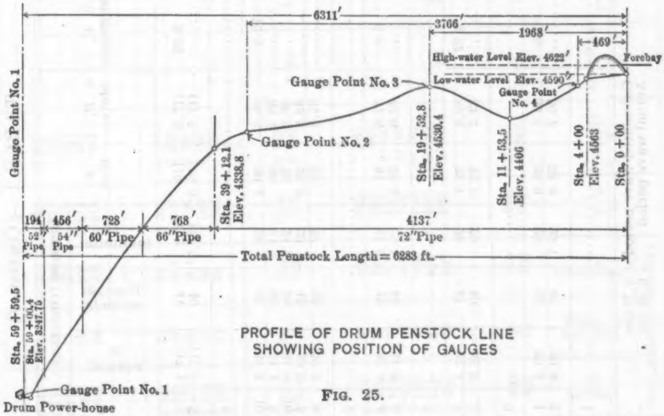
FIG. 24.

It is to be noted that charts were obtained for each and every test, but, in the foregoing, only the numbers of those charts which are published herein are given. These, however, are typical of the others, and, in fact, the similarity of the charts produced by a repetition of the same gate motion is remarkable.

A summary of the results of all the tests is given in Table 1.

Velocity.—The quantity of water flowing has been ascertained from the pressure readings at the nozzle by using the formula,

$$Q = A\sqrt{\frac{2\,g\,H}{\left(\frac{1}{C}\right)^2 - \left(\frac{A}{A_1}\right)^2}} \quad \text{.................(6)}$$



PROFILE OF DRUM PENSTOCK LINE
SHOWING POSITION OF GAUGES

FIG. 25.

and, from this value, the velocities at various points of the pipe line have been deduced.

Where A = area of the nozzle opening = 46.96 sq in. = 0.326 sq. ft.;

A_1 = area of the pipe leading to the nozzle = 405.4 sq. in. = 2.82 sq. ft.;

C = the coefficient of discharge = 0.96;

H = the pressure head at the entrance to the nozzle, as shown by Gauge No. 1.

Pipe Line.—Table 2 states the physical data of the pipe line.

The nozzle arrangement and the position of Gauge No. 1 are shown on Fig. 5. This line was of riveted steel.

TABLE 1—(*Continued*).

Experiment No.	Gauge No.	L, in feet.	Q, in second-feet.	V, at gauge.	P, static.	P flow.	Opening (Observed). P, minimum.	Time, in seconds.	P, normal.	p.	Closing (Observed). P, maximum.	Time, in seconds.	P, normal.	p.	Calculated values of p, in pounds.
13	2	3 766	94	3.32	117.5	116	112	41	117.1	5.1	122.5	53	117.4	5.1	
14	....	3 766	....	....	117.5	116.5	111.5	40	117.0	5.5	122	52	117.3	4.7	
Average	....	....	....	....	....	....	....	....	....	5.3	....	....	....	4.9	4.87
1	3	1 968	365	12.9	36.3	30.1	24	40	34.3	10.3	47	54	36 0	11.0	
2	....	....	....	....	36.4	30	24.9	40	34.4	9.5	46	50	35.9	10.1	
3	....	....	....	....	36.3	30	24.2	39	34.3	10.1	47	50	35.8	11.2	
15	....	....	....	....	32.8	26.8	23	45	30.3	7.3	41.5	53	32.4	9.1	
16	....	....	....	....	32.5	27	22	42	30.5	8.5	42	58	32.3	9.7	
Average	....	....	....	....	....	....	....	....	....	9.2	....	....	....	10.2	9.9
9	3	1 968	186	6.6	32.6	31	27.2	43	32.0	4.8	37.9	52	32.5	5.4	
10	....	....	....	....	32.6	30.5	26.5	43	31.8	5.3	38.8	50	32.5	6.3	
Average	....	....	....	....	....	....	....	....	....	5.1	....	....	....	5.8	5.6
13	3	1 968	94	3.32	32.4	32.1	29	41	32.3	3.3	36	53	32.4	3.6	
14	....	....	....	....	32.7	32.1	29	40	32.5	3.7	36	52	32.7	3.3	
Average	....	....	....	....	....	....	....	....	....	3.5	....	....	....	3.4	2.54
18	4	469	186	6.6	30.7	29.9	29	43	30.4	1.4	31.9	53	1.2	......	
19	....	....	....	....	30.7	30.0	29.1	43	30.4	1.4	31.9	48	1.2	......	
Average	....	....	....	....	....	....	....	....	....	1.4	....	....	1.2	......	1.2

Calculated wave period = 8.76 sec.
Actual wave period = 6.95 sec.

TABLE 1.—PRESSURE VARIATIONS IN DRUM PENSTOCK DUE TO OPENING AND CLOSING OF NEEDLE NOZZLES.

Experiment No.	Gauge No.	L, in feet.	Q, in second-feet.	V, at gauge.	P, static.	P, flow.	Opening (Observed). P, minimum.	Opening (Observed). Time, in seconds.	Opening (Observed). P, normal.	p.	Closing (Observed). P, maximum.	Closing (Observed). Time, in seconds.	Closing (Observed). P, normal.	p.	Calculated values of p, in pounds.
15	1	6 311	365	32.4	582	540	532	52	558	26	608	54	580	28	
16					582.5	540	534	45	564	30	612	54	580.5	32.5	
Average										28				30.25	37
6	1	6 311	186	33	581.5	564	560	40	575.6	15.6	597	45	579.4	17.6	
7					581.5	564	558	43	574.7	16.7	597	40	578.5	18.5	
8					581.5	564	558	43	574.7	16.7	598	38	578.0	20.0	
9					581.5	564	558	43	574.7	16.7	596	52	580.5	15.5	
10					581.5	564	558	45	574.2	16.2	598	48	579.7	18.3	
18					582	566	561	43	575.8	14.8	598	40	579.2	18.8	
19					582	566	560	42	576.0	16.0	599	48	580.4	18.6	
Average										16.1				18.2	19.05
13	1	6 311	94	33.4	581.5	573	570	41	578.5	8.5	592	53	581.0	11.0	
14					581.5	574	570	40	579.0	9	592	52	581.0	11.0	
Average										8.75				11.0	
1	2	3 766	365	12.9	120	109	99	41	116.2	17.2	140	52	119.3	20.7	
2					120.5	109	99	40	116.7	17.7	140	47	119.4	20.6	
3					121.0	109.5	100	39	117.2	17.2	142	58	120.5	21.5	
15					117.5	107	108	45	118.1	10.1	135	54	117.0	18.0	
16					117.5	107	99	42	113.8	14.7	136	60	117.3	18.7	
Average										15.4				19.9	18.9
6	2	3 766	186	6.6	116	113	107.5	40	115	7.5	126	45	115.6	10.4	
7					117	113.5	105	43	115.7	10.7	127.5	40	116.4	11.1	
8					116	113	105	43	114.9	9.9	126	38	115.4	10.6	
9					116	113	107	43	114.9	7.9	126	52	115.8	10.2	
10					116.5	113.5	105.5	45	115.3	9.8	126	48	116.2	9.8	
Average										9.2				10.4	9.72

TABLE 2.—PHYSICAL DATA OF PIPE LINE.

Type of pipe.	Diameter, in inches.	Thickness of metal, in inches.	Length, in feet.
Lap-riveted.	72	¼	1 714.6
" "	72	5/16	1 147
" "	72	⅜	480
Butt strap.	72	⅜	730
" "	72	7/16	65
" "	66	7/16	244
" "	66	½	123
" "	66	9/16	119
" "	66	⅝	103
" "	66	11/16	97
" "	66	¾	82
" "	60	¾	114
" "	60	13/16	153
" "	60	⅞	96
" "	60	15/16	98
" "	60	1	71
" "	60	1 1/16	97
" "	60	1⅛	99
" "	54	1 1/16	91
" "	54	1⅛	113
" "	54	1 3/16	117
" "	54	1¼	134.4
" "	52	1¼	194.17
Cast steel.	36	1½	18
" "	26	¾	33.25 aver.
" " taper.	26 to 20	1⅛	4
		Total length......	6 337.42

Fig. 25 is a diagrammatic profile of the line showing the positions of the gauges used to record the pulsation effects. The end of the pipe line at the forebay reservoir is wide open.

Magnitude of Pulsation.—A study of these charts developed the fact that the magnitude of the pulsation (that is, the wave pressure) was more nearly in accord with Professor Joukovsky's formula $\left(\frac{2\,L\,V}{T\,g}\right)$ than with some of those advanced more recently. It was observed, however, that although there was a rather close agreement for Gauge Points Nos. 2, 3, and 4, the formula did not accord at all with the results at Point No. 1. As Gauge Points Nos. 2, 3, and 4, were in such positions that the pipe above them was of uniform diameter, and the pipe above Gauge No. 1 was of varying diameter, it was decided to endeavor to obtain a formula applicable to a line of varying diameter.

By a study of the principles advanced by Professor Joukovsky, Formula 1 was developed for a pipe of varying diameter. It will be remembered that, after proving that the maximum possible wave pressure due to instantaneous gate closure was $\frac{a\,V}{g}$, Professor Joukovsky

assumed that a gate closure in finite time could be taken as made up of a succeeding series of partial instantaneous gate closures, in which case, the pressure would rise by increments at the rate of

$$\frac{\frac{a\,V}{g}}{T},$$

and that this pressure rise would continue for the time, T, or until a reflected wave from the reservoir would cut it short in a time, $\frac{2\,L}{a}$.

Assuming such to be the case for a pipe of uniform diameter, it must be evident that, in a pipe of varying diameter, any section of the line of larger diameter may be regarded as an imaginary reservoir, or a point of pressure relief for all points below it, or rather between it and the gates. For, although an instantaneous gate motion will produce a wave, $\frac{a_1\,V_1}{g}$, in magnitude, in a pipe of diameter, D_1, it will continue on through a pipe of diameter, D_2, with a magnitude only of $\frac{a_2\,V_2}{g}$.* Therefore, when a wave reaches the first point of increase in diameter in the line, a partial relief of pressure occurs, and a return relief wave is started back toward the gate, where it will arrive in the time, $\frac{2\,L_1}{a_1}$. Similarly, partial relief waves will be started at each point of change in the section of the pipe. Following this conception, it is evident that a pressure wave started at the gate will increase at the rate, $\frac{\frac{a_1\,V_1}{g}}{T}$, for a time, $\frac{2\,L_1}{a_1}$, when the first partial relief wave reaches the gate. Then, the pressure will continue to increase at the rate, $\frac{\frac{a_2\,V_2}{g}}{T}$, for a time, $\frac{2\,L_2}{a_2}$, etc., until, during a final period equal to,

$$\left\{T-\left(\frac{2\,L_1}{a_1}+\frac{2\,L_2}{a_2}+\frac{2\,L_3}{a_3}+\cdot\cdot\cdot+\frac{2\,L_{x-1}}{a_{x-1}}\right)\right\}$$

the rise is at the rate of $\frac{\frac{a_x\,V_x}{g}}{T}$. When the gate completes its motion, the

* See Professor Joukovsky's method of calculation of maximum pressure in paper by Miss O. Simin.

maximum pressure at the gate is reached. This maximum pressure will then evidently be,

$$h = \frac{a_1\,V_1}{g\,T}\cdot\frac{2\,L_1}{a_1} + \frac{a_2\,V_2}{g\,T_1}\cdot\frac{2\,L_2}{a_2} + \quad . \quad . \quad . \quad . \quad . \quad + \frac{a_{x-1}\,V_{x-1}}{g\,T}\cdot\frac{2\,L_{x-1}}{a_{x-1}}$$

$$+ \frac{a_x\,V_x}{g\,T}\left(T - \left\{\frac{2\,L_1}{a_1} + \frac{2\,L_2}{a_2} + \quad . \quad . \quad . \quad . \quad + \frac{2\,L_{x-1}}{a_{x-1}}\right\}\right) \ldots(7)$$

Now, let,

$$T - \left(\frac{2\,L_1}{a_1} + \frac{2\,L_2}{a_2} + \quad . \quad . \quad . \quad . \quad + \frac{2\,L_{x-1}}{a_{x-1}}\right) = \frac{2\,l}{a_x} \ldots\ldots(8)$$

producing a convention whereby, for the purpose of calculation, the pipe line is terminated at the last point from which an imaginary relief wave could return to the gate, arriving in just a time, T, when the gate completes its motion.

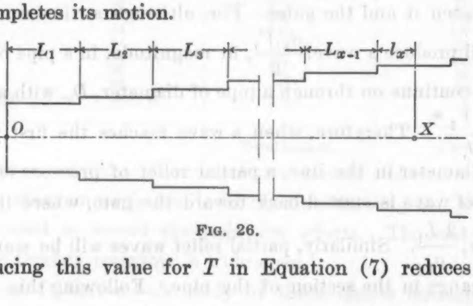


FIG. 26.

Introducing this value for T in Equation (7) reduces it to the form

$$h = \frac{2}{g\,T}\,(L_1\,V_1 + L_2\,V_2 + \quad . \quad . \quad . \quad . \quad + L_{x-1}\,V_{x-1} + l_x\,V_x) \ldots (1)$$

Having solved Equation (8), $L_1 + L_2 + L_3 + L_{x-1} + l_x$ will be the length of line to consider in fixing maximum pressure, and may be represented by OX in Fig. 26, where O represents the gate and X the farthest point from it. Note that, ordinarily, X will not be a point of change of pipe section, and that, ordinarily, l_x will not be a full length of section.

Referring to Equation (1), it is evident that, for slow gate closure, as T is less than t_0, X falls in the reservoir and V_x is 0, and the equation becomes

$$h = \frac{2\,(L_1\,V_1 + L_2\,V_2 + L_3\,V_3 + \quad . \quad . \quad . \quad \text{etc.})}{g\,T} \ldots\ldots\ldots(2)$$

in which all sections of the pipe are used for the full length of the

line. This is, then, the equation for slow gate closure with pipe of varying diameter.

For a pipe of uniform diameter with slow gate closure, Equation (2) reduces to the form,

$$h = \frac{2\, L_1\, V_1}{g\, T} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

the formula advanced by Professor Joukovsky.

Although these equations were deduced for pressure at the gate, it is evident that they will hold true for any point in the line, inasmuch as the first wave produced by gate closure at any point in the line is the same as though an imaginary gate were closed at this point in the same time, T, as the actual gate is closed. In reference to this, one should note carefully the definition of L_1, L_2, L_3, etc., as these vary for different points.

In Equations (1), (2), and (3), h is limited to a maximum value equal to $\frac{a_1\, V_1}{g}$, and this maximum will occur at some point in the line for all values of $T < \frac{2\, L_1}{a_1}$ and for all points in a line of uniform diameter with instantaneous gate closure.

In order not to lengthen this paper unduly, the writer will not discuss this limiting value and its application here, but will refer to his discussion of Mr. Warren's paper (previously mentioned) in which he has gone rather fully into the matter.

Equation (2) is the one applicable to the experiment described herein because of the slow gate closure. Equation (2) may also be deduced mathematically, as follows, using as a basis the proposition of mechanics that

Impulse = Force × Time = Change in Momentum.

The total change in momentum,

$$= \left[\frac{w\, A_1\, V_1\, L_1}{g} - \frac{w\, A_1\, v_1\, L_1}{g}\right] + \left[\frac{w\, A_2\, L_2\, V_2}{g} - \frac{w\, A_2\, v_2\, L_2}{g}\right] + \text{etc.} \quad \ldots\ldots\ldots (9)$$

or, where the change in momentum is measured from or to a complete state of rest, this becomes

$$\frac{W\, A_1\, L_1\, V_1}{g} + \frac{W A_2\, L_2\, V_2}{g} + \text{etc.} \quad \ldots\ldots\ldots\ldots (10)$$

as $v_1 = v_2 = v_3 = 0$.

The impulse, or product of force and time, would be $\frac{W A h}{2} \times T$, where A represents the area of the gate opening and $\frac{h}{2}$ is used as the average pressure head.

The assumption that the average force during the time of gate movement is equal to the maximum force divided by two would seem to be logical, in that, as the gate closes uniformly, it exposes to the flowing column a surface of impedance which varies uniformly from zero to a maximum. It would seem, therefore, that the force resisting the flow would vary in the same ratio.

Equating, we have,

$$\frac{W A h}{2} T = \frac{W A_1 L_1 V_1}{g} + \frac{W A_2 L_2 V_2}{g} + \text{etc.} \ldots \ldots (11)$$

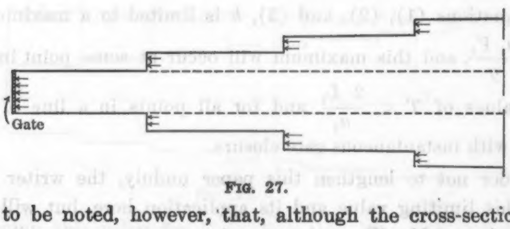


FIG. 27.

It is to be noted, however, that, although the cross-sectional area of the pipe may vary, only that portion of the water column having a cross-sectional area equal to that of the gate opening will be stopped directly by the pressure developed on the gate. (See Fig. 27.) The remainder of the resisting force will be developed at the points of change of diameter of the pipe by the increased pressure at these points.

This formula, therefore, becomes Equation (2)

$$h = \frac{2 (L_1 V_1 + L_2 V_2 + L_3 V_3 + \quad . \quad . \quad . \quad . \quad \text{etc.})}{g T} \ldots (2)$$

It might be noted that Mr. Warren, in using the impulse momentum principle to develop a pressure formula for slow gate closure, has assumed the time element for the impulse action to be equal to $\left(T + \frac{L}{a} \right)$, in place of T, in view of the fact that the water column is not completely brought to rest until a time, $\frac{L}{a}$, after the gate has finally completed its motion. This, however, the writer believes to be incorrect.

for the following reason: The first impulse increment or wave at the beginning of gate motion, although immediately affecting the water at the gate, would not affect the final portion of the water column at the reservoir until a time, $\frac{L}{a}$, after the gate had begun its motion. Similarly (as Mr. Warren points out), the effect of the last increment of force from the gate would not affect the portion of the water column at the reservoir until a time, $\frac{L}{a}$, after the gate had completed its motion, but, evidently, these two effects balance, and every portion of the water column has been acted on only for the time, T.

Normal Pressure Curve.—In interpreting the charts, it must be noted carefully that the value of h here referred to is the increased pressure due to pulsations only, and, therefore, measurements of h must be made with reference to the normal position of the hydraulic gradient at the same instant. As the gate moves, there is a gradual change in the normal position of the hydraulic gradient at that point (from flow to static position or its reverse), due to change in velocity head and friction losses.

Fig. 28 shows diagrammatically such typical normal pressure change, for opening and closing, respectively, at a point near the gate of the Drum pipe line. For purposes of comparison the normal line for closing (shown in the upper left part of Fig. 29) has been replotted on the right of one of the gauge charts.

In considering wave pulsations due to gate motion, they must be considered and measured with reference to such a normal curve. For instance, a wave form of Type 5, Fig. 30, superimposed in such a normal curve, would give the result shown at the right of Fig. 29. The writer thinks that this feature has sometimes been overlooked by investigators in the past, and the magnitude of the wave pressure has been considered with reference to static conditions rather than with reference to normal, as defined above.

Herein, wherever pressure is noted as above or below "normal" the writer refers to the pressure variation above or below the normal curve, as defined above, and all values of h are so taken.

Discussion of Results.—The last column of Table 1 shows the theoretical values of p for each experiment, where p, in pounds,

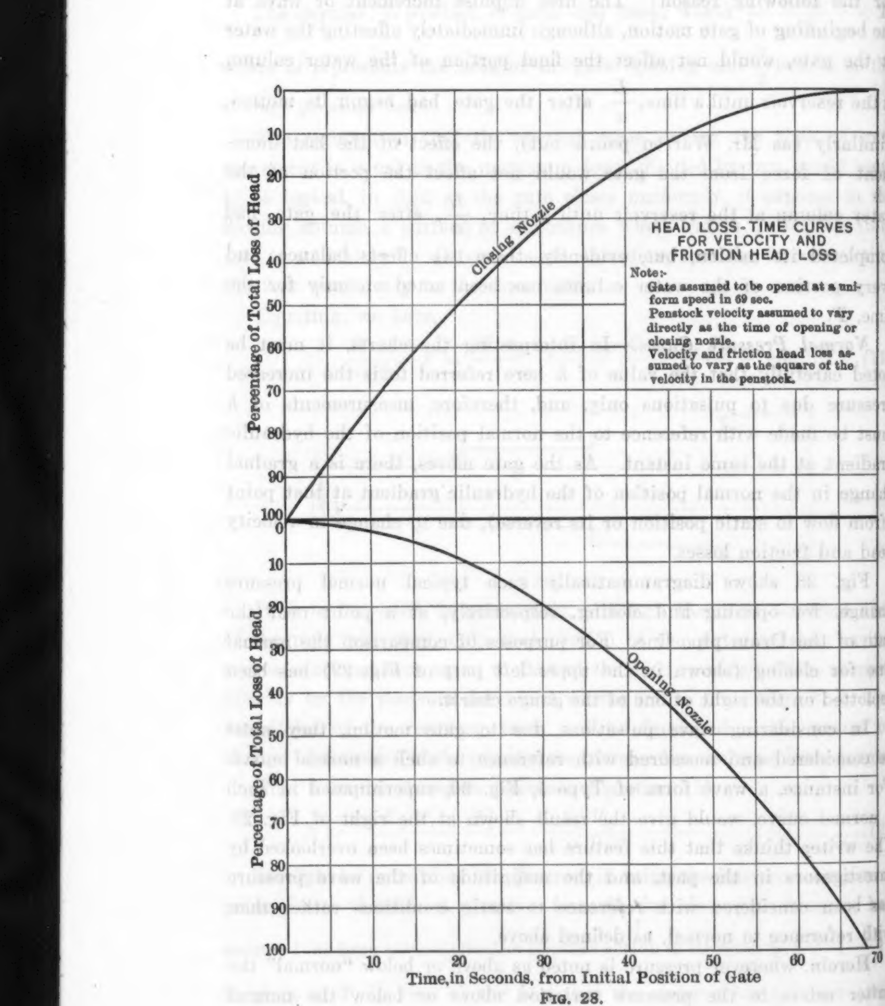

HEAD LOSS - TIME CURVES
FOR VELOCITY AND
FRICTION HEAD LOSS
Note:-
Gate assumed to be opened at a uniform speed in 69 sec.
Penstock velocity assumed to vary directly as the time of opening or closing nozzle.
Velocity and friction head loss assumed to vary as the square of the velocity in the penstock.
Closing Nozzle
Opening Nozzle
Percentage of Total Loss of Head
0
10
20
30
40
50
60
70
80
90
100
Time, in Seconds, from Initial Position of Gate
10
20
30
40
50
60
70

FIG. 28.

= 0.434 *h*, and where *h* has been calculated from Equation (2). The values of *h* have been converted into pounds for ready comparison with the gauge readings. In comparing such calculated values with the average of observed values, the agreement is thought to be remarkably close, both for opening and closing waves. These results certainly seem to substantiate the formula advanced, not only at the gate, but for any other point in the line, and for varying velocities. The time element shown in Table 1 is the time (after the gate begins its motion) when the maximum effect was observed, and this is noted in order that the correct position of the normal at the same instant may be taken.

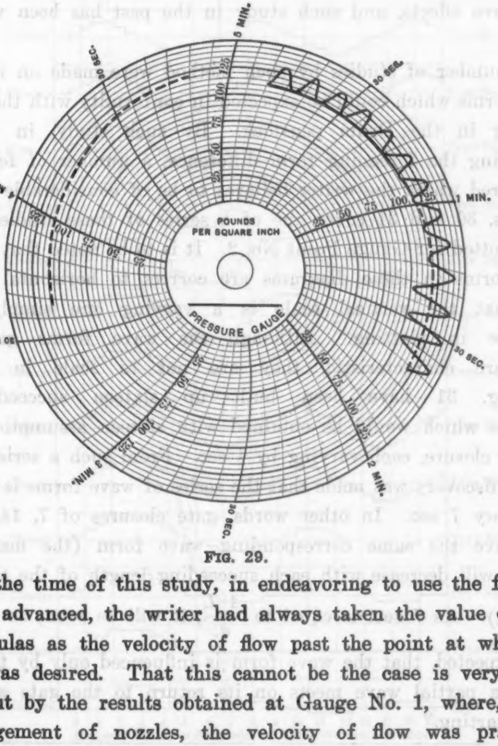


FIG. 29.

Up to the time of this study, in endeavoring to use the formulas heretofore advanced, the writer had always taken the value of *V* in such formulas as the velocity of flow past the point at which the pressure was desired. That this cannot be the case is very clearly brought out by the results obtained at Gauge No. 1, where, due to the arrangement of nozzles, the velocity of flow was practically constant; at the same time, the wave pressure can be seen to have

varied almost directly with the number of gates opened, or, in other words, with the velocity in the main body of the pipe line.

Wave Synthesis.—In reviewing the charts, it was noted at once that they were quite different in form for points away from the gate from any wave curves previously obtained by other investigators (so far as the writer knows). Previous wave forms, in general, have shown a period of normal pressure between each super-normal and sub-normal crest. In an endeavor to account for this, a study of probable wave forms was made. The writer has always felt that the synthetical method advanced by Professor Joukovsky (which is clearly explained by Miss Simin*) was very useful in studying probable wave effects, and such study in the past has been very helpful to him.

A number of studies by such method were made on the probable wave forms which could be expected in conformity with the conditions existing in the Drum penstock. By such study, in addition to explaining the foregoing form difference, a number of features were discovered which the writer believes to be of considerable interest.

Figs. 30 and 31 cover one of a series of these studies, and have been plotted for Gauge Point No. 2. It is to be noted that the varying wave forms in these diagrams are correct to horizontal scale only, and that the vertical scale is a varying one simply for convenience in plotting. Although the wave forms given, therefore, are characteristic, they are not to scale in magnitude. In Fig. 31 have been built up sixteen succeeding forms of wave which would be obtained with sixteen assumptions of time of gate closure, each varying by ½ sec. From such a series the interesting discovery was made that the series of wave forms is a recurring one every 7 sec. In other words, gate closures of 7, 14, or 21 sec. will have the same corresponding wave form (the magnitude, of course, will decrease with each succeeding length of the time of gate opening). As 7 sec. is equal to $\frac{4\,L}{a}$, it will be seen, as might have been expected, that the wave form is influenced only by the phase in which a partial wave meets on its return to the gate a new wave "just starting."

* *Proceedings*, Am. Water Works Assoc., 1904, p. 341.

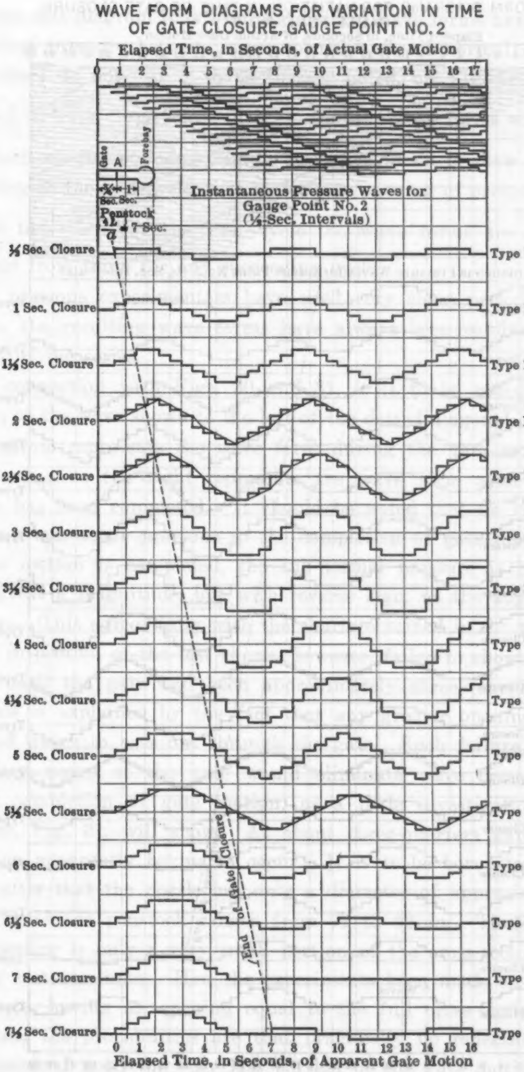

WAVE FORM DIAGRAMS FOR VARIATION IN TIME OF GATE CLOSURE, GAUGE POINT NO. 2
Elapsed Time, in Seconds, of Actual Gate Motion
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
Gate
A
Forebay
Instantaneous Pressure Waves for Gauge Point No. 2 (¼ Sec. Intervals)
Penstock
7 Sec.
½ Sec. Closure
1 Sec. Closure
1½ Sec. Closure
2 Sec. Closure
2½ Sec. Closure
3 Sec. Closure
3½ Sec. Closure
4 Sec. Closure
4½ Sec. Closure
5 Sec. Closure
5½ Sec. Closure
6 Sec. Closure
6½ Sec. Closure
7 Sec. Closure
7½ Sec. Closure
Type No. 2
Type No. 3
Type No. 4
Type No. 5
Type No. 6
Type No. 7
Type No. 8
Type No. 7
Type No. 6
Type No. 5
Type No. 4
Type No. 3
Type No. 2
Type No. 1
Type No. 2
End of Gate Closure
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
Elapsed Time, in Seconds, of Apparent Gate Motion

FIG. 30.

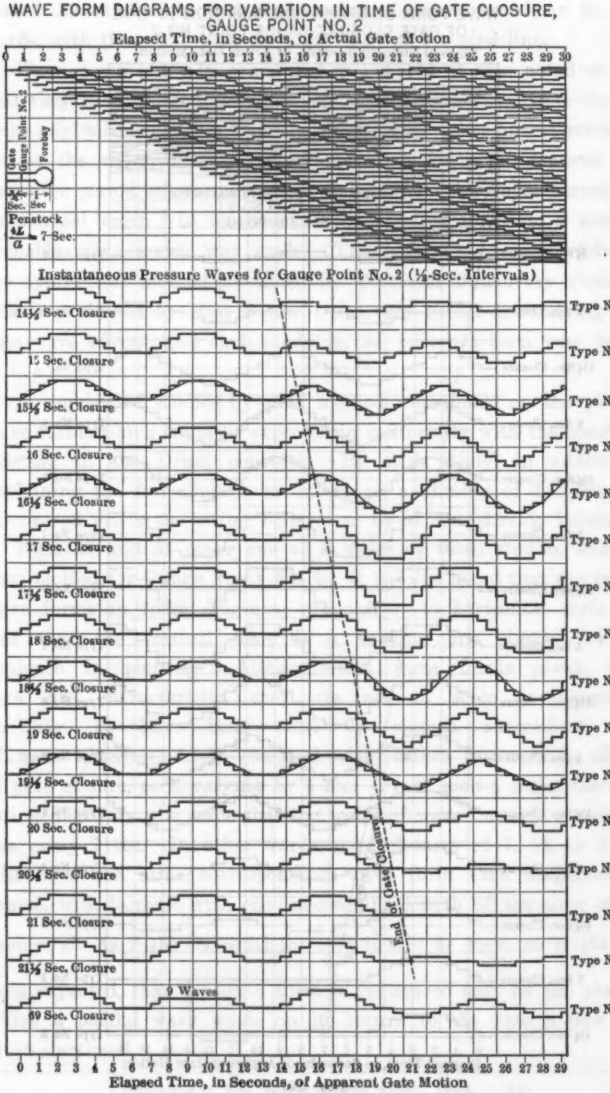

WAVE FORM DIAGRAMS FOR VARIATION IN TIME OF GATE CLOSURE, GAUGE POINT NO. 2
Elapsed Time, in Seconds, of Actual Gate Motion
Gate
Gauge Point No. 2
Forebay
Penstock
$\frac{4L}{a}$ = 7 Sec.
Instantaneous Pressure Waves for Gauge Point No. 2 (½-Sec. Intervals)
14½ Sec. Closure
15 Sec. Closure
15½ Sec. Closure
16 Sec. Closure
16½ Sec. Closure
17 Sec. Closure
17½ Sec. Closure
18 Sec. Closure
18½ Sec. Closure
19 Sec. Closure
19½ Sec. Closure
20 Sec. Closure
20½ Sec. Closure
21 Sec. Closure
21½ Sec. Closure
9 Waves
69 Sec. Closure
Type No. 2
Type No. 3
Type No. 4
Type No. 5
Type No. 6
Type No. 7
Type No. 8
Type No. 7
Type No. 6
Type No. 5
Type No. 4
Type No. 3
Type No. 2
Type No. 1
Type No. 2
Type No. 3
End of Gate Closure
Elapsed Time, in Seconds, of Apparent Gate Motion

FIG. 31.

From this diagram it is seen that certain wave forms have a period of normal pressure between super-normal and sub-normal crest, and that others do not. It is to be further noted, on Fig. 30, that, for a period of gate closure less than $\frac{2L}{a}$, all the wave forms which occur have such normal periods. Fig. 30 has been drawn to show the forms occurring in the first cycle, that is, for gate opening of periods ranging from 0 to $\frac{4L}{a}$. For the first cycle the wave forms are somewhat different from those for any succeeding cycle.

As previous experimenters have used very short periods of gate closure, the resulting wave forms have always been confined to this portion of the series.

In connection with Figs. 30 and 31, it is to be noted that the portion of the wave shown to the left of the dotted diagonal line across the diagram represents the wave form during the gate motion, and that portion to the right represents the wave form after the gate motion has been completed. It should be noted that no sub-normal pressures can occur previous to the completion of gate motion; but, after a motion is completed, the sub-normal pressure is practically of as great a magnitude, but with reverse sign, as the super-normal pressure. This agrees fully with the charts obtained in the tests.

An inspection of the test charts, however, failed to show any wave-crests until the gate had been approximately three-quarters closed, and this is explained by the fact that any greater opening permits reflected waves to pass out through the gate. Such failure to reflect secondary waves at the gate would eliminate wave crest previous to the completion of gate motion, as a little investigation of the diagram, Fig. 31, will show. At about three-quarters gate motion, reflection apparently begins to occur. It is to be borne in mind in this matter that the nozzle has only a diameter of approximately 8½ in., whereas the penstock ranges from 72 to 52 in. Thus, the full gate opening is only a very small portion of the cross-section of the moving water column. Had the experiments been made on the ordinary gate, having an opening equal to the full cross-section of the pipe line, the probabilities are that practically no reflections would have occurred until the very last portion of the gate motion. Such results would give an apparent wave form similar to that on which

Mr. Warren has based his conclusions. Although there is no limit in magnitude (except that for instantaneous gate closure, $\frac{aV}{g}$) for the portion of the wave formed during the gate motion, the studies herein submitted will show that certain ratios of length of penstock to time of closure should tend to damp and reduce both the magnitude and the number of the waves after the gate closure has been completed. When it is seen—as by the charts—that the pulsations may last from 5 to 10 min. in the ordinary penstock, it will be at once apparent that the ultimate destructive effect on any line might be importantly reduced by choosing a proper time of gate closure.

Below the sixteen forms studied in Fig. 31, is shown the form for the actual gate period, 69 sec. This corresponds to Wave Type 3. Although this might apparently be considered nonconformative to the actual chart wave, it will be noted, from the curve of gate opening, that the gate does not close quite uniformly, but has an increasing rate of closure at the end which might draw the wave into a different phase from that which would otherwise be expected. A final rate of gate motion corresponding to a time of opening of 68 sec. should apparently give a wave form corresponding to Wave Type 5, Fig. 31, which is more nearly what was actually obtained.

From this it would seem that by a slight adjustment of the time of gate opening between any 65 and 71 sec., some better form of wave might be obtained in the Drum line than that actually now existing.

Amplitude of Vibration.—Finally, a study was made to ascertain, if possible, how closely the formula for velocity of wave propagation

$$a = \frac{12}{\sqrt{\frac{W}{g}\left(\frac{1}{K} + \frac{D}{E\,b}\right)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(5)$$

could be made to apply to riveted pipe lines. Accordingly, the varying values of a_1, a_2, a_3, etc., were calculated.

In using the formula in connection with the riveted pipe, it was necessary to make allowance for the effect of joints and laps on the rigidity of the pipe, and therefore on the speed of wave propagation, a. Allowance was made for roundabout laps and butt straps by adding the total length of such joints parallel to the axis of the pipe and calculating the time element for an equal length of pipe having

a thickness equal to double the nominal thickness of pipe for lap-riveted work, and for the butt-strap material, using a thickness equal to the nominal thickness plus the thickness of the roundabout strap.

To compensate for the extra rigidity and resistance to diametrical stretch of the longitudinal joint, the additional quantity of metal in tension was computed as uniformly distributed around the pipe, that is, an average thickness, taken over and above nominal, was used.

The results obtained by such method gave a period of 8.76 sec., as compared with a very uniform wave length of 6.95 sec. given by the charts. Although this did not give as close an agreement as the writer had hoped for, the discrepancy is probably due to the fact that nominal instead of actual diameters and thicknesses of pipes were used. This wave period has in no wise entered into the foregoing tables or calculations, but was calculated as a matter of interest. It would be of use in the case of a rapidly moving gate, as entering into the maximum possible value of h.

In conclusion, the writer again calls attention to the fact that the results of these experiments show the importance of allowing for water-hammer in the design of pipe lines, and that, even for slow-moving gates, the matter cannot be safely overlooked. Finally, he hopes that others interested in this subject will discuss these results and pass judgment on them.

DISCUSSION

Mr. Gibson.

NORMAN R. GIBSON,* M. AM. SOC. C. E. (by letter).—This paper is one of the most interesting and instructive that have come to the notice of the writer, who, during the past year, has had occasion to make a careful study of the subject. The results of the experiments are evidently reliable, and they supply the much needed data with which analytical calculations may be verified. With many of the conclusions of the author the writer is in accord, but not with regard to the general application of his formula proposed for pressure variation from normal. In the writer's opinion, Mr. Vensano's error lies chiefly in the second assumption in the note on the curve shown in Fig. 28, namely, "penstock velocity assumed to vary directly as the time of opening or closing nozzle"; and this assumption, apparently, has led to an incorrect interpretation of the data obtained from his experiments. The close agreement between the calculated values of p and the values obtained by the experiments is due to the fact that the head acting on the nozzle was very high, in which case the retardation of the flow of water in the pipe was more nearly uniform than it would have been under a lower head. To apply the author's formula to pipes under comparatively low head, however, would lead to results which it is doubtful would be confirmed by experiment.

The influence of the net head on the flow of water in pipes, and the consequent rise of pressure caused by gradually stopping the flow, has been made the subject of a paper prepared recently by the writer, in which formulas are derived from Joukovsky's formula, $h = \frac{V a}{g}$, for instantaneous water-hammer. This paper at the present time is undergoing revision, by request, to make it more easily understood, and it is hoped will soon be in shape for publication. The treatment of the subject as contained therein is too long to be introduced here as a discussion of Mr. Vensano's paper, but the results of his experiments are so closely in accord with the writer's analytical results that it is hoped a brief explanation of them will be found helpful in studying the data.

The influence of the net head on the phenomena that occur in a pipe when the flow is being arrested cannot be neglected, because the rate of retardation of the flow depends on two factors: first, the closing of the gate, and second, the head acting on the orifice at the gate. In other words, the velocity of the water in the pipe depends on the area of the gate opening and the net head acting on that opening. The magnitude of the rise of pressure caused by gradually closing the gate depends on the velocity destroyed, the rate at which it is destroyed, the length of the pipe line, the compressibility of water, and the elas-

* Niagara Falls, N. Y.

ticity of the pipe. The last two factors are taken into consideration in the velocity of the pressure wave, a, as determined by Joukovsky, with which all students of this subject are familiar. For an instantaneous closing of the gate, the magnitude of the pressure rise depends only on the velocity destroyed, the compressibility of water, and the elasticity of the pipe. Mr. Gibson.

The relation between the velocity in the penstock, the area of the gate opening, and the net head, may be expressed by the equation,

$$V = B\sqrt{H} \quad \text{......................} (1)$$

where V = velocity of flow in pipe of uniform diameter, in feet per second;

H = net head acting on the orifice, in feet;

B = simply a number representing the gate opening.

During the closing of the gate, all three of these quantities vary. V and B become zero, and H rises to its maximum value. The rate at which B varies is known from the movement of the gate. The rate at which V varies is unknown, and depends on the increase in the value of H, which is also unknown. At all times during the gate opening, however, the relation of the three variables, as expressed by the equation, must be maintained.

Joukovsky's formula for the rise of pressure caused by an instantaneous closing of the gate in a pipe is

$$h = \frac{V\,a}{g},$$

where h = rise of pressure, in feet;

a = velocity of the pressure wave, in feet per second;

g = acceleration due to gravity.

For a slight instantaneous change in velocity,

$$\Delta h = \frac{a}{g}\,\Delta V \quad \text{......................} (2)$$

The determination of the value of a has been explained by the author, and need not be repeated here. As already stated, the derivation of the writer's formulas for the rise of pressure caused by the gradual closing of turbine gates, is too long to be given here, and the formulas themselves are too complex to be used without a lengthy explanation. Using Equations (1) and (2), however, a numerical problem may be solved by the trial and error method of arithmetic integration for the conditions under which Mr. Vensano's experiments were performed. The calculated results may then be compared with those obtained by experiments.

It is unfortunate that the pipe line described by the author is of varying diameter, as this complicates the problem at the outset, and no doubt accounts to some extent for the difference he found between

Mr. Gibson.

the calculated and observed values of the wave period. The observed value of the wave period, namely, 6.95 sec., will be used in the following calculations, and from it is found the value of a, as $\frac{4\,L}{a} = 6.95$ sec.

Therefore, $a = \frac{4 \times 6337.42}{6.95} = 3\,647$ ft. per sec.

After determining the value of a by calculation or experiment, it would appear reasonable to assume that the pipe was of uniform diameter, such that the velocity in it was a weighted mean of all the velocities in the various sections with respect to their length. That is to say, the sum of the products, $L_1\,V_1$, $L_2\,V_2$, $L_3\,V_3$, etc., divided by the total length of the pipe, would give a mean velocity, which, if gradually destroyed, would cause the same pressure rise as would be caused by the destruction of the various velocities in the pipe as it actually existed. Such an assumption may be questioned, but it simplifies the labor of making the calculations, and in this case, at least, seems to be justified by the results.

Table 3 presents the pipe data, from which the mean velocities were thus found to be:

15.055 ft. per sec. when $Q = 365$ cu. ft. per sec.
7.672 " " " " $Q = 186$ " " " "
3.877 " " " " $Q = 94$ " " " "

TABLE 3.—PIPE DATA. $Q = 365$.

Division.	Diameter, in inches.	Area, in square feet.	Velocity. V	Length. L	LV
1	20-26	2.93	33.30	4.0	133.2
2	26	3.687	24.740	33.25	825.0
3	36	7.068	25.818	18.0	464.7
4	52	14.748	24.740	194.17	4803.7
5	54	15.904	22.950	455.4	10451.5
6	60	19.635	18.589	728.0	13532.9
7	66	23.758	15.363	768.0	11798.8
8	72	28.274	12.909	4136.6	53400.2
				6337.42	95410.0

For $Q = 365$ Mean $V = \frac{95\,410}{6337.42} = 15.055$ ft. per sec.
For $Q = 186$ Mean $V = \quad = 7.672$ " " "
For $Q = 94$ Mean $V = \quad = 3.877$ " " "

The calculations to determine the rise of pressure for the conditions under which the author's Experiments Nos. 15 and 16 were performed, are tabulated in Table 4. These may be explained as follows:

The static head with gate closed was 582 lb. = 1 341 ft. The gauge reading at the gate was 540 lb. = 1 244 ft. To this must be added

the velocity head, due to the velocity at the gate, which was 32.4 ft. per sec. The velocity head, therefore, was 16.3 ft., and the total net head producing discharge through the orifice was 1 260.3 ft. The head lost in friction under the full flow was 1 341 — 1 260 = 81 ft. The value of B may now be found from Equation (1), Mr. Gibson.

$$B = \frac{V}{\sqrt{H}} = \frac{15.055}{\sqrt{1\,260}} = 0.4241.$$

TABLE 4.

(1) Interval.	(2) Gate. B	(3) Head. H	(4) Velocity. V	(5) $118\,\Delta V$ Δh	(6) $\Sigma\,\Delta h$	(7) $0.3574\ V^2$ h_f	(8) $\Delta\,h_f$	(9) $\Sigma\,(\Delta h + \Delta h_f)$
0	0.4241	1260.0	15.055			81.0		
			0.155	17.5				
1	0.4168	1279.2	14.90		17.5	79.3	1.7	19.2
			0.29	32.8				
2	0.4084	1279.8	14.61		15.3	76.5	4.5	19.8
			0.28	31.7				
3	0.3995	1284.0	14.33		16.4	73.4	7.6	24.0
			0.33	37.3				
4	0.3895	1291.8	14.00		20.9	70.1	10.9	31.8
			0.37	41.8				
5	0.3784	1295.5	13.63		20.9	66.4	14.6	35.5
			0.42	47.5				
6	0.3657	1305.3	13.21		26.6	62.3	18.7	45.3
			0.49	55.4				
7	0.3513	1311.8	12.72		28.6	58.0	23.0	51.8
			0.53	59.8				
8	0.3354	1319.0	12.19		31.0	53.0	28.0	59.0
			0.60	67.8				
9	0.3180	1329.8	11.59		36.8	48.0	33.0	69.8
			0.65	73.5				
10	0.2991	1334.9	10.94		36.7	42.8	38.2	74.9
			0.72	81.3				
11	0.2781	1348.2	10.22		44.6	37.4	43.6	88.2
			0.84	94.8				
12	0.2544	1359.7	9.38		50.2	31.5	49.5	99.7
			0.92	104.0				
13	0.2285	1369.2	8.46		53.8	25.6	55.4	109.2
			0.97	109.5				
14	0.2009	1376.6	7.49		55.7	20.1	60.9	116.6
			1.07	121.0				
15	0.1719	1391.6	6.42		65.3	14.7	66.3	131.6
			1.15	130.0				
16	0.1411	1395.8	5.27		64.7	9.9	71.1	135.8
			1.21	136.8				
17	0.1083	1407.2	4.06		72.1	5.9	75.1	147.2
			1.33	150.1				
18	0.0725	1416.3	2.73		78.0	2.7	78.3	156.3
			1.365	154.2				
19	0.0368	1416.5	1.365		76.2	0.7	80.3	156.5
			1.365	154.2				
20	0.	1419.0	0.		78.0	0.	81.0	159.0

The rate at which B varies is not uniform, as is shown by the author's curve on Fig. 6. From this curve the time of gate closure is shown to have been about 69 sec. During that time the pressure wave started by the first movement of the gate would travel up to the forebay and back at the rate of 3 647 ft. per sec., and, therefore,

Mr. Gibson. would make 19.88 return trips. For the sake of simplicity, assume that 20 return trips were made. It should be remembered that the velocity of the pressure wave is constant and independent of the magnitude of the wave. Now assume that the gate was closed in 20 instantaneous movements, each movement occurring regularly every $\frac{2\,L}{a}$ seconds. The time, $\frac{2\,L}{a}$, will be called one "interval."

The value of B at each movement of the gate may now be determined from the author's curve in Fig. 6, by dividing the time base into 20 even spaces and multiplying 0.4241 by the ratio of the area of nozzle opening opposite each division to the full area of nozzle opening. The result should be plotted on a curve, and, after smoothing out the irregularities, the values of B may be taken off and recorded, as shown in Column 2 of Table 4.

Table 4 may now be completed as follows: In the first line opposite $B = 0.4241$, set down in Columns 3, 4, and 7 the known values of H, V, and h_f (friction head). Next assume a trial reduction in velocity, caused by the initial instantaneous movement of the gate, and set the figure down in Column 4 under the value of V, and subtract it from V, placing the difference immediately underneath. This trial figure is ΔV, and is assumed to be destroyed instantaneously by the first movement of the gate. A pressure wave, $\Delta\,h$, of magnitude $= \frac{(\Delta\,V)\,a}{g}$ $= (\Delta\,V)\,\frac{3\,647}{32.2} = 113\,\Delta\,V$, is therefore started up the pipe. The product of 113 ΔV is set down in Column 5, opposite ΔV. In Column 6 is recorded the algebraic sum of the values of Δh. In Column 7 the total friction head, due to the velocity shown in Column 4, is set down, and in Column 8, the friction head, recovered at each operation, is shown. For convenience, h_f may be made equal to $C\,V^2$, in which C is a coefficient obtained from the known values at the beginning. In the example, $C = 0.3574$. Column 9 shows the sum of the opposite items in Columns 6 and 8. Having obtained the figure in Column 9, it is added to the net head, $H = 1\,260$, and the sum is set down in the next lower line in Column 3. The result must now be checked to see that $B\,\sqrt{H} = V$, where B is now 0.4168, and H and V are the values opposite. If the relation is not satisfied, a new trial value of ΔV must be chosen, and the operations repeated until a check is obtained. After trial, the initial value of ΔV was found to be 0.155. The operations for obtaining the figures on the third line are not quite the same as those just described, because, after assuming the next trial value of ΔV, and multiplying it by 113, it must be remembered that the resulting pressure at the gate is reduced by the return of the first wave which has traveled up to the forebay and back, and now

changes to sub-normal and repeats the journey. The time at which the gate is given its second movement, has been selected purposely to coincide with the return of the first wave. The item opposite Interval 2, in Column 6, therefore, is the difference between the first Mr. Gibson.

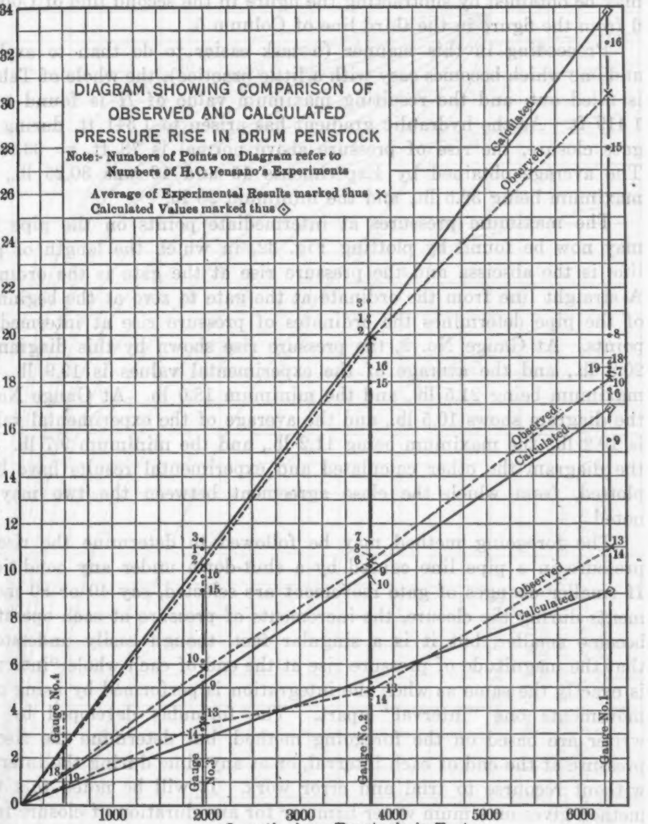


FIG. 32.

two figures in Column 5. The other operations are similar to those already described, and the resulting figures in Columns 3 and 4 are checked similarly with the value of $B = 0.4084$. The succeeding lines are filled in by the same process, always keeping in mind the return of the preceding waves, and whether they change to sub-normal or

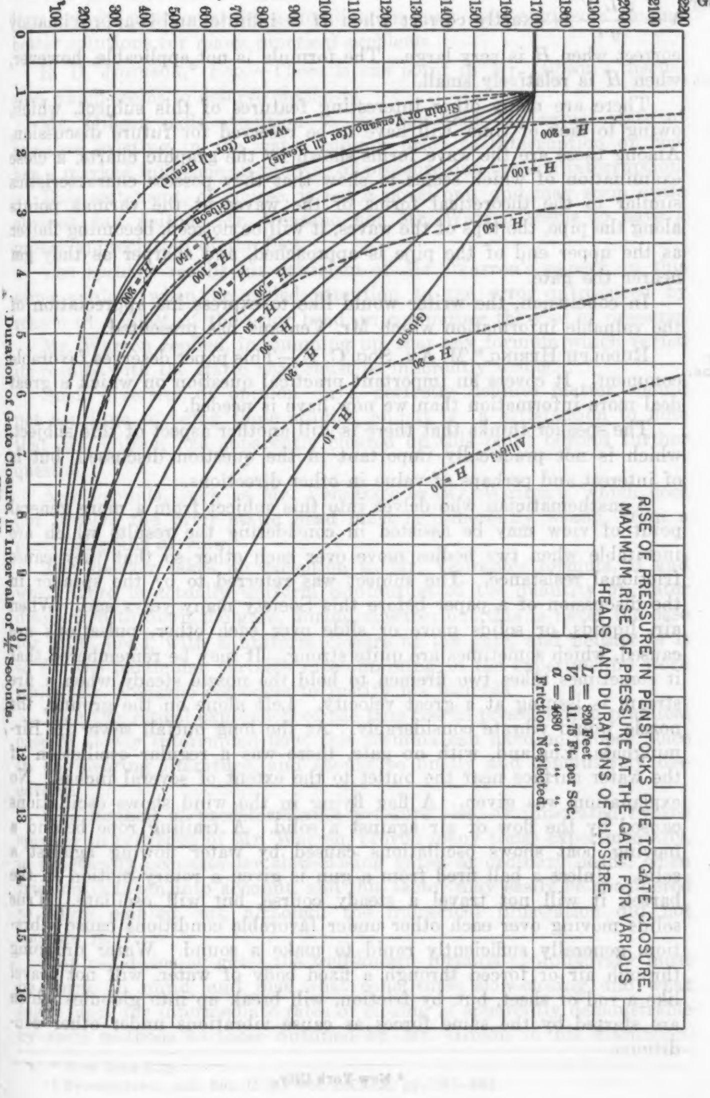
Mr. Gibson. super-normal. A little study will disclose the fact that the figure in the second line of Column 6 may be obtained by subtracting the figure in the first line of Column 6 from the figure in the second line of Column 5. Similarly, the figure in the third line of Column 6 may be obtained by subtracting the figure in the second line of Column 6 from the figure in the third line of Column 5.

Proceeding in this manner (a task easier to do than to explain, and one which becomes easy with a little practice), the whole of Table 4 is filled out, and the resulting maximum value of H is found to be 1 419 ft. As the hydraulic gradient has arisen to 1 341 ft. during the gate closure, the rise of pressure above normal is 78 ft. = 33.85 lb. The average obtained by Experiments 15 and 16 was 30.25 lb., the maximum being 32.5 lb., and the minimum 28 lb.

The maximum pressures at intermediate points on the pipe line may now be found by plotting Fig. 32, in which the length of pipe line is the abscissa and the pressure rise at the gate is the ordinate. A straight line from the ordinate at the gate to zero at the beginning of the pipe determines the ordinates of pressure rise at intermediate points. At Gauge No. 2, the pressure rise shown by this diagram is 20.2 lb., and the average of the experimental values is 19.9 lb., the maximum being 21.5 lb., and the minimum 18.0 lb. At Gauge No. 3, the diagram shows 10.5 lb., and the average of the experimental values is 10.2 lb., the maximum being 11.2 lb., and the minimum 9.7 lb. On the diagram the other calculated and experimental results have been plotted, from which the close agreement between the two may be noted.

The foregoing method may be followed to determine the rise of pressure in a pipe line caused by a shut-down under any conditions. If smaller changes of gate movement are selected, say 40 or 80 movements during the closure, the increments of pressure at each operation become smaller, but it is a singular fact, though easily understood, that the magnitude of pressure rise at the end of each whole "interval" is exactly the same as when the integration is performed by using gate movements one "interval" apart. The formulas developed by the writer are based on the foregoing method, but determine the rise of pressure at the end of each interval, or at any time during the interval, without recourse to trial and error work. It will be noted that this method gives maximum water-hammer for any duration of closure from zero up to 1 "interval," $\left(\frac{2\,L}{a}\right)$. As the duration of closure becomes longer, the resulting rise of pressure obtained by the method finally approaches the value given by Allievi's well-known formula, the curves having the general characteristics shown in Fig. 33. On this figure the corresponding curves obtained from the formulas of Allievi, Vensano, and Warren are shown for comparison.

Mr. Gibson.


RISE OF PRESSURE IN PENSTOCKS DUE TO GATE CLOSURE.
MAXIMUM RISE OF PRESSURE AT THE GATE, FOR VARIOUS HEADS AND DURATIONS OF CLOSURE.
L = 820 Feet
V = 11.75 Ft. per Sec.
a = 4680 " "
Friction Neglected.
Rise of Pressure, in Feet.
Duration of Gate Closure, in Intervals of 2L/a Seconds.
Warren (for all Heads)
Strain or Vensano (for all Heads)
Gibson
Allievi
H = 200
H = 100
H = 50
H = 10

Mr. Gibson.

The writer's conclusion, therefore, is that Mr. Vensano's formula, $h = \frac{2 L V}{g t}$ is exactly correct when H is infinite, and is approximately correct when H is very large. The formula is not applicable, however, when H is relatively small.

There are many other interesting features of this subject, which, owing to lack of time, will have to be reserved for future discussion. Among these are the wave forms shown by the graphic charts, a close examination of which seems to show that they possess characteristics similar to the theoretical forms of the waves at the various points along the pipe, the tops of the waves, it will be noticed, becoming flatter as the upper end of the pipe is approached, and sharper as they get nearer the gate.

In conclusion, the writer would like to express his appreciation of the valuable information which Mr. Vensano has presented.

Mr. Hering.

RUDOLPH HERING,* M. AM. SOC. C. E.—This paper deserves favorable comment. It covers an important practical question on which a great deal more information than we now have is needed.

The speaker thinks that there is still another aspect of this subject, which is not practically important in the question discussed, but is of interest and perhaps of value in other directions.

A mathematician who delves into this subject from a more general point of view may be assisted in considering the results which are inevitable when two bodies move over each other so that they cause frictional resistance. The subject was referred to by the speaker in the discussion of a paper before this Society many years ago. When air, liquids, or solids move or slide over each other, pulsations are caused, which sometimes are quite strong. It may be remembered that it sometimes takes two firemen to hold the nozzle steady when a fire stream is issuing at a great velocity. Left alone on the ground, the nozzle would vibrate considerably. At the long outfall sewer in Birmingham, England, with no gate, there was a regular oscillation of the water surface near the outlet to the extent of several inches. No explanation was given. A flag flying in the wind shows oscillations caused by the flow of air against a solid. A trailing rope behind a moving boat shows oscillations caused by water flowing against a solid. Unless a ball fired from a gun is given a rotary motion in the barrel, it will not travel a steady course, but will oscillate. True solids moving over each other under favorable conditions cause vibration, generally sufficiently rapid to make a sound. Water dropping through air or forced through a fixed body of water, will not travel like a rod or sheet, but, by friction, will break up into globules which are started by the same forces as cause vibrations under other conditions.

* New York City.

The speaker believes that this is not only an interesting subject, but that its mathematical generalization may help engineers to find better solutions for many practical problems. Mr. Hering.

R. D. JOHNSON,* ESQ.—There is one point in Mr. Gibson's discussion which seems to deserve emphasis. Mr. Johnson.

Mr. Gibson calls attention to the fact that the actual state of pressure existing in the water column prior to interruption of flow has a marked influence on the rise of pressure due to such interruption.

This statement, of course, is not a novel one, and may seem quite obvious, but neither Mr. Vensano nor Mr. Warren seems to have a proper appreciation of the fact.

The speaker recalls his criticism of Mr. Warren's paper† on this same subject when he called attention to the error introduced by reason of neglect of the prior head, in computing the rise of pressure. Mr. Warren replied, in summing up, that any formula which varied its results with the static head must be inherently wrong.

Such diversity of opinion on a perfectly definite matter should not continue to befog these studies, and it is a satisfaction to note that Mr. Gibson has apparently cleared up this point beyond further question.

Any formula, such as Mr. Vensano's or Mr. Warren's, which does not involve the prior static head must be inherently wrong, from a practical point of view.

The speaker takes no exception to Mr. Vensano's formula, if and when there is actually uniform diminution of the quantity of water discharged through the gate, during closure; but this is never the case for finite head, with a uniform closure of the gate, and neither is it so for the particular variable rate obtaining in the experimental work which Mr. Vensano considers to justify his formula.

As a theoretical expression of the true rise in pressure in a practical case of slowly interrupted flow, this formula can have no proper place in engineering literature, and should be finally and conclusively discarded.

The method of predetermining pressure rise, as illustrated by the speaker in criticism of Mr. Warren, proven many times experimentally, is practically accurate for slow-closing gates, at ordinary heads, when friction is taken into account, and this factor may easily be considered as there pointed out, although the numerical illustration did not include it.

For rapid-closing gates the well-known formula of Joukovsky must be borne in mind, and just what constitutes slow-closing and what takes place for intermediate rates of closing is apparently demonstrable by such methods as those outlined by Mr. Gibson in his discussion.

* New York City.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, pp. 277–281.

Mr. Johnson.

If the static head affecting Mr. Vensano's experiments, or the manner of gate motion during closure, had, either or both, been materially different, the rise of pressure would probably not so nearly have agreed with the formula, $2\frac{L V}{g T}$, in fact, it could not, except by a coincidence of conditions, and this agreement which he finds must be regarded as purely accidental.

Mr. Warren.

MINTON M. WARREN,* ASSOC. M. AM. SOC. C. E. (by letter).—As far as the writer knows, Mr. Vensano's excellent paper is the first to present data obtained from reliable tests on slow closing of gates in a pipe line.

As the writer stated in the closing discussion of his paper on the same subject,† the formula, $h = \frac{2 L V}{g T}$, if backed by experimental graphs, would have a very strong case. Although Mr. Vensano's data do strengthen this case considerably, they do not prove it, for the following reasons:

1.—All the results are obtained from one pipe line where, owing to varying pipe size, nozzle conditions, etc., the agreement of results with the formula may easily be an accident.

2.—In the pipe line in question, the fact that the nozzle, which was closed, was very small in comparison with the pipe size, makes a special condition which, by aiding wave reflection, would tend to increase the water-hammer.

3.—The formula contains no term affected by the period of the wave in the pipe, which must have an effect on the water-hammer, especially in comparatively quick closing of gates.

The reason for the variation of these results from those obtained with the writer's formula, $h = \frac{L V}{g\left(T - \frac{L}{a}\right)}$, is due to these special conditions, and the violation of the first assumption on which this formula is based. If used properly, this formula still seems to be the best for general use.

In deducing his formula by impulse and momentum, Mr. Vensano uses the whole mass, but uses the time, T, at the end of which the upper end of the mass is still moving. The writer believes that $T - \frac{L}{a}$ should be used, as explained in his paper.

The formula, $h = \frac{L V}{g T}$, and Allievi's formula—which Mr. R. D. Johnson also advocates—are absolutely wrong and should be dis-

* Cambridge, Mass.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 305.

carded once and for all. The absurdity of the latter was pointed out by the writer in a letter* in which it was shown to give impossible results, in some cases several thousand per cent. too high. Mr. Vensano's formula is at least on the safe side, and never more than 100% high. Mr. Warren.

H. C. VENSANO,† M. AM. SOC. C. E. (by letter).—The writer desires to thank Messrs. Gibson, Hering, Warren, and Johnson for their criticisms. As Mr. Johnson's criticism is of the destructive variety, it is not very helpful. His contention, that the agreement between the formula advanced and the experimental work is merely a coincidence, is scarcely to be taken seriously, especially, when the writer adds that in a previous set of experiments the time of gate opening was about 50 sec. Due to the fact that the recording apparatus was not registering properly, the charts were not preserved, but the results agreed quite well with the subsequent work. In these later experiments, varying times of gate opening would have been used had it been possible, but the plant was then operating to produce power, and the time of gate opening could not be changed. Mr. Vensano

Mr. Gibson's criticism, on the other hand, is constructive, and is therefore of more interest. Due to the brief way in which Mr. Gibson has advanced his theory, the writer feels unable to comment on it. He will, meanwhile, take great interest in Mr. Gibson's coming paper.

It is to be noted, however, that though Mr. Gibson's mathematical illustration apparently agrees with the experimental results obtained by the writer, his diagram does not show that this would be the case. Unfortunately, it has not been extended to cover his mathematical illustration, but it is very evident that, with heads higher than 200 ft., his curves will approach Mr. Warren's very closely. Mr. Warren's curve gives results only about one-half as great as those obtained by the writer's formula and corroborated by his experiments.

Mr. Gibson's chief objection to the writer's formula is based on the fact that the assumption was made that "the penstock velocity was assumed to vary directly as the time of opening or closing the nozzle". The writer realized a slight weakness in his deductions, because of this very assumption, and purposely introduced Fig. 6 to show the degree of approximation involved in it. However, referring to this diagram, it will be noted that, during the opening of the nozzle, the ratio of nozzle opening to time is a constant, but the total change of nozzle velocity is only about 5 per cent. In other words, for the nozzle opening, this assumption is very close to the truth. For the nozzle closing, this is not so true, due to some variation in rate of gate motion, as shown by the diagram, but this change is very slight for the first 30 sec. of the gate motion. A study of the surges shown by the opening

* *Engineering News-Record*, August 16th, 1917, p. 320.
† San Francisco, Cal.

Mr. Vensano. diagrams and also a study of the first 30-sec. period of the curve of the closing diagram will show that the writer's formula is accurate for these conditions, and would tend to prove that this assumption has very little bearing on the total result.

Mr. Gibson also makes a statement that his work is based on the work of Professor Joukovsky and takes into account all the factors considered by that writer, the apparent implication being that the writer's work did not take into account all such factors. This is not the case, and, as a matter of fact, all the writer's ideas were based on those of Joukovsky, and all like factors were considered and included in the formula advanced.

Mr. Warren, in his third objection to the paper, also states that the formula cannot be correct, as it "contains no term affected by the period of the wave in the pipe". Although the final formula does not include such a term directly, the period of the pipe has been taken into account, as will be noted in Equation 7, in which it will be seen that each portion of the right-hand side contains a term of the form, $\frac{2\,L}{a}$. These terms, as a whole, take into account the period of the pipe, and, in the final equation, they have been eliminated by cancellation and change of form, but the period, nevertheless, has been taken into careful consideration as limiting the rise of pressure.

Although the views expressed by Messrs. Gibson and Johnson are apparently considerably at variance with the writer's, some explanation of the differences may be possible. It is remarkable, sometimes, how different viewpoints lead to apparently widely different results. As Mr. Johnson apparently agrees with Mr. Gibson, and as both are interested in problems of power generation, it is possible that their research work has been with a view to voltage and speed control of electric generating apparatus. Also, in view of the fact that Mr. Gibson's diagrams deal only with low heads (200 ft. head being considered very low in Western practice), it is probable that the experiments to which they refer were made in connection with turbine installations; in other words, with a turbine acting as the pipe gate.

The writer's experiments, on the other hand, were made with a view to studying "water-hammer" as affecting the design of pipe lines. An impulse-wheel rather than a turbine installation was chosen for experimentation, for the reason that the small nozzle (gate) opening compared with the cross-section of the pipe line would give almost complete reflection of partial pressure waves at that end of the pipe. This condition of complete reflection would give the simplest and truest condition of water-hammer. It will be noted that, neglecting the 20- and 36-in. pipe, which is actually part of the nozzle body, the

area of opening is only 2.2% of the cross-section of the pipe line for one nozzle opened, or about 9% with four nozzles opened. Mr. Vensano.

In using a turbine as the gate control, one would ordinarily have a very large proportionate gate opening, practically oftentimes the full area of the pipe line. Under such circumstances, no reflection of partial pressure waves would be obtained until the gate was almost closed. A study of the writer's diagrams, Figs. 30 and 31, would indicate that, without reflection of partial pressure waves, the maximum pressure might rise to a higher value than given by his formula. This would in no wise minimize the usefulness of the formula submitted, but would represent a difference in physical conditions similar to that produced if sound-wave experiments were being performed on an open organ pipe, rather than on one with a closed end. The results would naturally be at variance.

The writer believes that future experiments should include a study of that ratio of gate opening to pipe cross-section at which reflection of partial pressure waves begins. For openings above such value, under which conditions secondary wave effects are doubtless produced, some extension of the formula submitted would, no doubt, have to be made, and these results would perhaps explain the high values advanced by Messrs. Gibson and Johnson, which they claim to be corroborated by low-head practice.

In conclusion, the writer is sorry that Mr. Johnson feels that the formula advanced "has no place in engineering literature," and believes that decision to be somewhat hasty. He trusts that future experiments will prove conclusively its usefulness.

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Paper No. 1407

AIR TANKS ON PIPE LINES

By Minton M. Warren, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Irving P. Church, R. D. Johnson, P. Wahlman, L. R. Jorgensen, and Minton M. Warren.

Synopsis.

The object of this paper is to call attention to the utility of air tanks on long pipe lines and to some of their practical disadvantages, to formulate the theoretical principles involved, and to derive simple formulas which may be used as a guide to determine their safe dimensions.

The contents of this paper may be briefly classified as follows:

Formulas for air tank design;

Derivation of formulas;

Practical questions of design and operation; and

Numerical examples.

Conclusions.—Although there is a general idea that air tanks are not successful for regulating purposes on pipe lines, and that their design is a matter of great uncertainty, it can be shown that, if properly built, they are of great practical value in improving regulation and preventing water-hammer, and their design is simple and based on fundamental laws.

The literature on the design of air tanks on pipe lines for waterwheel regulation is very limited. This is, perhaps, due to two causes: first, because of a general belief that the problem is so complex as to need a difficult mathematical solution by calculus; second, because

of a popular idea that, in order to be effective, the air tank must be so large as to be prohibitive in cost. Neither of these, however, is true. The design of such a tank is simple, and comparatively small tanks are in commercial operation on pipe lines, and have greatly improved the regulation and reduced the trouble from water-hammer.

For the sake of convenience the formulas are given first, the assumptions used in their derivation are then listed, and finally they are derived from Newton's second law of motion and the physical laws governing the expansion and compression of air.

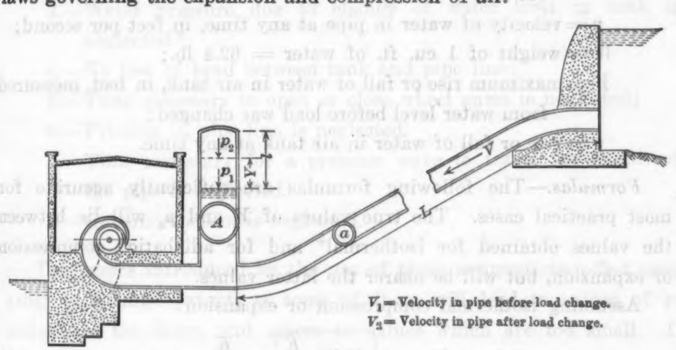

FIG. 1.

More complicated formulas are appended which are derived by calculus with fewer assumptions.

Nomenclature.—The following gives the meaning of every term used in deriving the formulas. All units are in feet, pounds, or seconds.

A = cross-sectional area of air tank, in square feet;

g = acceleration of gravity = 32.2 ft. per sec. per sec.;

$K = \frac{L\,a}{g\,A\,p_1}(V_1 - V_2)^2$;

K_1 = Head lost between air tank and pipe line, in feet, when $a\,(V_1 - V_2)$ cu. ft. per sec. are flowing into tank;

L = length of pipe, in feet, between open reservoir or fore-bay and air tank;

l = length of air column in tank before load change, in feet;

a = cross-sectional area of pipe, in square feet;

p_1 = air pressure in air tank before load change, including atmospheric pressure, measured in feet of water;

p_2 = maximum or minimum air pressure in air tank due to load change, including atmospheric pressure, measured in feet of water;

t = time for pressure in air tank to change from p_1 to p_2, in seconds;

V_1 = velocity of water in pipe before load change, in feet per second;

V_2 = velocity of water in pipe after load change, in feet per second;

v = velocity of water in pipe at any time, in feet per second;

W = weight of 1 cu. ft. of water = 62.4 lb.;

Y = maximum rise or fall of water in air tank, in feet, measured from water level before load was changed;

y = rise or fall of water in air tank at any time.

Formulas.—The following formulas are sufficiently accurate for most practical cases. The true values of Y and p_2 will lie between the values obtained for isothermal* and for adiabatic* compression or expansion, but will be nearer the latter values.

Assuming isothermal compression or expansion:

$$Y = \sqrt{K\,l + \frac{K^2}{4}} \mp \frac{K}{2} \qquad \text{(A)}$$

$$\text{where } K = \frac{L\,a}{g\,A\,p_1}\,(V_1 - V_2)^2$$

$$p_2 = \frac{p_1\,l}{l \mp Y} \qquad \text{(B)}$$

Assuming adiabatic compression or expansion:

$$(l - Y)^{1.41} = \frac{Y\,l^{1.41}}{K + Y} \qquad \text{(C)}$$

for loads thrown off, and

$$(l + Y)^{1.41} = -\frac{Y\,l^{1.41}}{K - Y} \qquad \text{(C}_1\text{)}$$

for loads thrown on.

$$p_2 = p_1\left(\frac{l}{l \mp Y}\right)^{1.41} \qquad \text{(D)}$$

* Isothermal compression or expansion assumes that the temperature of the air in the tank remains constant. All the heat generated in compressing the air, therefore, must escape instantly through the walls of the tank and into the water.

Adiabatic compression or expansion assumes that no heat passes through the tank walls or between the air and water.

Where plus and minus signs appear, the minus sign applies to loads thrown off, the plus sign to loads thrown on.

Assumptions.—These formulas are derived by using the following assumptions, and, before using them on any specific case, these assumptions should be examined in order to make sure that they apply to the case in question.

1.—Pressure in tank rises or falls at a constant rate;*

2.—Water level in tank rises or falls at a constant rate;*

3.—Water pressure due to change of water level in tank is neglected.*

4.—No loss of head between tank and pipe line;

5.—Time necessary to open or close wheel gates is neglected;

6.—Friction in pipe line is neglected.

7.—Time necessary for a pressure wave to travel the length of the pipe is neglected.

8.—Governor action is neglected.

The errors introduced by the use of these assumptions offset each other to a large extent, as some of them will lead to values of p_2 which are too large, and others to values which are too small. It should be noted, however, that if the loss of head between the tank and pipe line is comparatively large, the pressure may rise higher in the pipe line and wheel than in the tank.

In general, the error caused by Assumption 6 is negligible, as the rise in air pressure is large in comparison with the difference in friction head at any two loads. In order to be absolutely safe, this friction head may be added to the value of p_2 obtained from the formulas for loads thrown off, but this will give results which are too large. The reverse is true for loads thrown on.

Assumption 7 is reasonable, as the time for a rise of pressure to be transmitted along the pipe line is usually small in comparison with the time of oscillation in the air tank.

The effect of governor action (Assumption 8) is discussed later.

From numerical computations it appears that Assumptions 1, 2, and 3, taken together, lead to values of p_2 which are too small.

* Formulas (P) and (Q), derived by calculus, are appended; they are deduced without the use of Assumptions 1, 2, and 3. These may be used for the final design of a tank.

Derivation of Formulas.—To avoid confusion, the formulas for load thrown off (air compressed) are first derived.

Using the fundamental law,

$$\text{Mass} \times \text{Acceleration} = \text{Force},$$

and multiplying through by time, we get

$$\text{Mass} \times \text{Change in Velocity} = \text{Force} \times \text{Time} \ldots\ldots (E)$$

The mass of the water in the pipe is $\frac{W\,a\,L}{g}$. When a load is thrown off, the velocity is cut down from V_1 to V_2, and the change of velocity, therefore, is $(V_1 - V_2)$.

The force tending to slow down the water in the pipe is due to the difference in head at the two ends, or rise of pressure at the lower end. Assuming no head lost between the air tank and pipe line (Assumption 4), and neglecting the pressure due to the rise of water in the tank (Assumption 3), this difference in head is equal to the rise of air pressure in the tank, which varies from zero at the beginning to $(p_2 - p_1)$ at the end. Assuming the rise to be at a constant rate (Assumption 1), the average force tending (during the time, t) to slow down the water will be $W\,a\,\frac{p_2 - p_1}{2}$, in pounds.

As a matter of fact, the average force is less than this, as it rises faster at the end than at the beginning.

Using Formula (E), and inserting the foregoing values, we get

$$\frac{W\,a\,L}{g} \times (V_1 - V_2) = W\,a\,\frac{p_2 - p_1}{2} \times t \ldots\ldots\ldots (G)$$

The time, t, is unknown. Neglecting the short time necessary for the governor to move the gates and decrease the quantity of water supplied to the wheel (Assumption 5), the net quantity of water coming into the tank when the load is first thrown off will be the volume flowing in from the pipe line $(a\,V_1)$ less the volume flowing out to the wheel $(a\,V_2)$. Therefore the water will start to rise in the tank at a velocity of $\frac{a}{A}\,(V_1 - V_2)$, in feet per second, and this will decrease to zero in the time, t. Assuming this decrease to be at a constant rate (Assumption 2), the average velocity will be half of this, or $\frac{a}{2\,A}\,(V_1 - V_2)$. As a matter of fact, the average velocity is

greater than this, for the velocity decreases at a slower rate at the end of the time than at the beginning.

If the water rises in the tank at an average velocity of $\frac{a}{2A}(V_1 - V_2)$, in feet per second, it will take $\frac{2\,A\,Y}{a\,(V_1 - V_2)}$ seconds to rise a distance, Y. Therefore,

$$t = \frac{2\,A\,Y}{a\,(V_1 - V_2)} \dots\dots\dots\dots\dots\dots (H^*)$$

Inserting this value of t in Formula (G), we get

$$\frac{W\,a\,L}{g}(V_1 - V_2) = W\,a\,\frac{(p_2 - p_1)}{2}\,\frac{2\,A\,Y}{a\,(V_1 - V_2)}.$$

Simplifying,

$$(p_2 - p_1)\,Y = \frac{L\,a}{g\,A}(V_1 - V_2)^2 \dots\dots\dots\dots\dots (I)$$

From this, Formulas (A) or (C) can be deduced by using the laws of thermodynamics which apply to the compression or expansion of gases. In order to get the relation between the two unknown quantities, p_2 and Y, it is necessary to know the variation in temperature in the air tank.

If the compression took place very slowly, and all the heat generated had time to escape into the water and through the walls of the tank, the isothermal relation would hold, and

$$\frac{p_2}{p_1} = \frac{\text{original volume of air in tank}}{\text{volume of air in tank after compression}} = \frac{A\,l}{(l - Y)\,A}$$

or

$$p_2 = \frac{p_1\,l}{l - Y} \dots\dots\dots\dots\dots\dots\dots (B)$$

If the compression took place very quickly, and all the heat of compression were retained, the adiabatic relation would hold, and

$$\frac{p_2}{p_1} = \left(\frac{l}{l - Y}\right)^{1.41} \dots\dots\dots\dots\dots\dots (D)$$

As a matter of fact, the true condition will be somewhere between the two. With full load thrown off, the isothermal assumption will give results which are much too small, and in general the adiabatic assumption is much nearer the truth.

---

* This formula, by itself, will give a value of t which is too large. See Formula (R).

For instance, in one actual case, for full load thrown off, the compression would take less than 5 sec., and, assuming adiabatic compression, the temperature would rise more than 100° Fahr. It is apparent that in this short time not much of the heat would escape, and the isothermal assumption, therefore, gives results about 14% too small.

Although formulas for isothermal compression give values of p_2 (maximum pressure) which are too small, they give larger values of Y (rise of water in the tank) than formulas for adiabatic compression.

Isothermal Compression.—Assuming, first, isothermal compression: Combining Formulas (I) and (B),

$$\frac{L\,a}{g\,A}(V_1 - V_2)^2 = p_1\left(\frac{l}{l - Y} - 1\right)\,Y = \frac{p_1\,Y^2}{l - Y} \quad \ldots\ldots (J)$$

let

$$K = \frac{L\,a}{g\,A\,p_1}(V_1 - V_2)^2 \quad \ldots\ldots (M)$$

and insert in Formula (J).

$$K = \frac{Y^2}{l - Y}$$

$$K\,l - K\,Y = Y^2$$

solving

$$Y = \sqrt{K\,l + \frac{K^2}{4}} - \frac{K}{2} \quad \ldots\ldots (A)$$

Adiabatic Compression.—Assuming, second, adiabatic compression: Combining Formulas (I) and (D),

$$\frac{L\,a}{gA}(V_1 - V_2)^2 = p_1\left[\left(\frac{l}{l - Y}\right)^{1.41} - 1\right]Y \quad \ldots\ldots (N)$$

Inserting K, as before,

$$K = \left(\left(\frac{l}{l - Y}\right)^{1.41} - 1\right)Y$$

or

$$(l - Y)^{1.41} = \frac{Y\,l^{1.41}}{K + Y} \quad \ldots\ldots (C)$$

This must be solved by trial and error. As a first trial, a value of Y slightly smaller than that obtained from Formula (A) may be used.

Load Thrown On.—In the case of a load thrown on, exactly the same reasoning applies, and Y, being measured down instead of up, will be a negative quantity. When the numerical value is inserted, the signs, therefore, must be changed as indicated.

Full Load Changes.—When full load is thrown off, the same formulas apply, and, as V_2 equals zero, Formula (I) becomes

$$(p_2 - p_1)\ Y = \frac{L\,a\,V_1^2}{g\,A}.$$

This can be checked easily by applying the theorem of work and energy, using the assumptions previously listed. All the water is stopped, and all the energy of the moving water is expended in compressing the air in the air tank.

Work of compressing air in tank = Energy of water in pipe line.

$$Y \times \frac{W A\ (p_2 - p_1)}{2} = W\,a\,L \times \frac{V_1^2}{2\,g}.$$

Simplifying,

$$(p_2 - p_1)\ Y = \frac{L\,a\,V_1^2}{g\,A}.$$

Similarly, for full load thrown on, V_1 becomes zero.

Practical Questions of Design, Surges, Etc.—In actual practice it has sometimes happened that an air tank made the regulation and surges in the pipe line worse instead of better. This has been due to surges which were set up and either aggravated by governor action or continued naturally for a considerable time before being damped out by friction.

These surges can be broken up by using some sort of a differential device which restricts the entrance to the tank, or a valve which causes the water to flow into the tank at a different rate from that at which it flows out.

Mr. R. D. Johnson has invented and patented a device which not only eliminates surges, but cuts down the size of the tank required.

In one hydro-electric development where trouble was caused by surges, a flap-valve surrounded by orifices was placed in the entrance to the air tank. When load was thrown on, the flap opened and allowed the water to flow out freely. When the water started to flow back into the tank, the flap closed and the water was forced through the orifices, with a consequent loss of head. This broke up the surges, and there was no more trouble.

In one case, under normal operating conditions, an air tank of moderate dimensions placed near the power-house decreased the sudden variations of pressure due to changing load by about two-thirds.

In order to keep the tank full of air, it is necessary to replenish the supply from time to time from a compressor or other source, as a certain quantity is apt to be absorbed by the water or leak through the tank. If the air should entirely leak out, the tank would not only become useless for regulating purposes, but might fail on account of dangerous pressures due to water-hammer.

The larger the volume of air in the tank, the more efficient will it be for regulating purposes. Care must be taken, however, not to fill the tank so full of air that when a sudden load is thrown on, all the water in the tank will be exhausted and the wheel will suck air. Automatic safety devices may be put on to prevent this, and gauges on the side of the tank will indicate the water level. It is conceivable that in some cases air might be given out by the water and fill the tank too full. The writer knows of no such case, however, although the reverse is common.

In most air tanks the compression due to load changes will take place in so short a time that little of the heat will have time to flow through the walls of the tank or into the water, and for this reason the formulas for adiabatic compression will give results nearer the truth. In any case, they are more conservative to use.

Other Formulas.—In order to get formulas without the use of Assumptions 1, 2, and 3, it is necessary to use the calculus. The other assumptions (4, 5, 6, 7, and 8) are used.

The net quantity of water flowing into the tank is equal to that flowing in minus that flowing out; therefore,

$$A \frac{d\,y}{d\,t} = a\,(v - V_2)$$

$$\text{Mass} \times \text{Acceleration} = \text{Force}$$

therefore (for load thrown off),

$$\frac{W\,a\,L}{g} \times \frac{d\,v}{d\,t} = -(p - p_1 + y)\,W\,a.$$

Using these two formulas in conjunction with Formulas (*B*) and (*D*) (the isothermal and adiabatic relation), we can get a differential equation for $\frac{d\,y}{d\,t}$ in terms of y and $d\,y$ which can be integrated by using

the fact that when $y = 0$, $\frac{dy}{dt} = \frac{a}{A}(V_1 - V_2)$; and when $y = Y$, $\frac{dy}{dt} = 0$.

By this integration we get the following:

Isothermal.

For load off:

$$Y^2 - 2p_1\left(l \log_e\left(1 - \frac{Y}{l}\right) + Y\right) = p_1 K \ldots\ldots\ldots\ldots (P)$$

For load on:

$$Y^2 - 2p_1\left(l \log_e\left(1 + \frac{Y}{l}\right) - Y\right) = p_1 K \ldots\ldots\ldots (P_1)$$

Adiabatic.

For load off:

$$Y^2 - 2p_1\left[\frac{l}{0.41} - \frac{l^{1.41}}{0.41(l - Y)^{0.41}} + Y\right] = p_1 K \ldots\ldots (Q)$$

For load on:

$$Y^2 - 2p_1\left[\frac{l}{0.41} - \frac{l^{1.41}}{0.41(l + Y)^{0.41}} - Y\right] = p_1 K \ldots.(Q_1)$$

A formula for t, derived by calculus, using all the assumptions except 2, is:

$$t = \frac{\pi}{2}\sqrt{\frac{L\,A\,Y}{a\,g\,(p_2 - p_1)}} \ldots\ldots\ldots\ldots\ldots (R)$$

This t is the time required for Y to reach a maximum. It is the time for a quarter of the cycle, and, after the water has risen in the tank (with load thrown off), it will, like a pendulum, fall to the starting point again, and on below it to almost the same extent; then rise again, and so on until damped out by friction. It is to prevent this surge that valves or differential devices must be used. Formula (R) will give a value of t which is too small. The true value will be between this and the value obtained from Formula (H).

Where it is desired to compute the heat generated or lost by adiabatic compression or expansion, the following formula may be used.

$$\frac{T_2}{T_1} = \left(\frac{l}{l \mp Y}\right)^{0.41}.$$

The minus sign is for load off, the plus sign for load on.

T_1 is the absolute temperature before start of rise or fall;

T_2 is the absolute temperature at maximum point of rise or fall.

If the Fahrenheit scale is used, the absolute temperature may be obtained by adding 459° to the temperature registered on the ordinary thermometer.

Head Lost in Entering Air Tank.—If the pipe line runs into the air tank on one side and out on the other, the pressure in the pipe line will at all times be equal to the pressure in the air tank (Assumption 4).

When a load is suddenly thrown off, the quantity of water flowing out of the tank will be suddenly decreased, but the quantity flowing in will not be changed until the water has risen and the air pressure consequently increased. This increase of pressure causes a difference of head, between the two ends of the pipe line, which slows down the water. It increases from 0 to $(p_2 - p_1)$ and is greatest at the time when the water has been slowed down to its new velocity.

It is obvious that the air tank would be more efficient if the pressure could be made to rise instantly and act on the water at the time when it is most needed, or before the velocity has been checked.

By causing a loss of head between the air tank and the pipe line, this condition can be approached. If the loss of head is made too great, however, the pressure may rise instantly to a greater value than it would have reached with the simple tank, and this is all the more undesirable as the sudden rise causes water-hammer, and is more difficult for the governors to handle than the gradual rise.

A well-designed orifice, therefore, would cause about the same loss of head at the beginning as the compressed air would cause at the end.

An approximate formula can be worked out by computing the maximum loss of head between the pipe line and the air tank at the start of the rise. This will be

$$C \frac{(V_1 - V_2)^2}{2g},$$

where C is a constant depending on the shape and size of the entrance to the tank.

Calling this loss of head K_1, and assuming that the difference in head varies at a constant rate from this value at the beginning to $(p_2 - p_1)$ at the end, we get an average force of $\frac{K_1 + p_2 - p_1}{2}$, in pounds, tending to slow down the water.

Inserting this value in Formula (G) and proceeding as before, we get for isothermal compression:

$$(K_1 - p_1)\, Y^2 - (Kp_1 + K_1\, l)\, Y = -\, Kp_1\, l \ldots\ldots. (S)$$

For load thrown on, this becomes

$$(K_1 + p_1)\, Y^2 + (K_1\, l - Kp_1)\, Y = Kp_1\, l \ldots\ldots. (S_1)$$

Where $K_1 = 0$, these become Formula (A).

A similar formula can be worked out for adiabatic compression. When the calculus is used and Assumptions 1, 2 and 3 are omitted, a differential equation is derived which cannot be integrated.

Considering the many uncertainties of the problem, the foregoing formula is probably sufficiently accurate, but it should be used with the full realization of the assumptions made in its derivation.

Numerical Example.—In order to show the comparative variation of the different formulas, the following numerical case has been worked out for full load thrown off and on. The dimensions are taken from an actual plant, and an extreme velocity of 12.5 ft. per sec. is taken to illustrate more clearly the differences in the various formulas.

$l = 52$ ft.;

$L = 2\,100$ ft.;

$a = 44.2$ sq. ft. (pipe 7.5 ft. in diameter);

$A = 77$ sq. ft. (two tanks 7 ft. in diameter);

$p_1 = 470$ ft.

$V_1 = 12.5$ ft. per sec.;

$V_2 = 0.$

Table 1 shows the values obtained from the various formulas. In studying these results, it should be noted that the difference in the results of the formulas for adiabatic and isothermal compression is 14 per cent. The difference between the results obtained by the more approximate Formulas (A) and (C) and the results obtained from Formulas (P) and (Q) is from 3 to 6% for this case.

TABLE 1.

	Formula.	Y, in feet.	p_2, in feet.	Percentage of normal pressure (p_1).
Full load off:				
Isothermal...............	(A) and (B)	20.0	764	163
	(P)	20.8	783	166
Adiabatic................	(C) and (D)	16.9	818	174
	(Q)	17.7	845	180
Full load on:				
Isothermal...............	(A) and (B)	32.5	290	62
	(P_1)	27.5	307	65
Adiabatic................	(C_1) and (D)	27.6	258	55
	(Q_1)	23.8	276	59

For load thrown on, Formulas (P_1) and (Q_1) give results 6% apart for the two relations; and the more approximate Formulas (A) and (C_1) give results differing from Formulas (P_1) and (Q_1) by from 3 to 4 per cent.

The error caused by assuming either the adiabatic or isothermal relation, therefore, may be greater than the error caused by using the simpler Formulas (A) and (C) and so, taking into account the other assumptions, it would seem that Formulas (A) and (C) are sufficiently accurate for most practical cases, although a final check may be made by Formulas (P) and (Q).

If a restricted orifice were inserted between the pipe line and air tank which caused a loss of head of 190 ft. with the full load flow ($a\ V_1$) entering the tank the results would be as follows:

	Formula	Y	p_2	Percentage of p_1
Full load off: ($K_1 = 190$)	(S)	15.3	667	142
Full load on: ($K_1 = 190$)	(S_1)	18.7	346	74

A comparison of these figures with those given for the simple tank shows that the use of an orifice makes the air chamber a great deal more effective. It also reduces the possibility of trouble from surges.

The foregoing formulas have not been published before, as far as the writer knows, with the exception of Formula (P) which was deduced by Mr. Johnson in the article mentioned on the next page.

In studying the effect of differential devices and friction in air tanks and surge tanks, the following references will be found useful:

"The Surge Tank in Power Plants." By R. D. Johnson. *Transactions*, Am. Soc. Mech. Engrs., Vol. 30 (1908), page 833.

"The Differential Surge Tank." By R. D. Johnson. *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), page 760.

"Penstock and Surge Tank Problems." By Minton M. Warren. *Transactions*, Am. Soc. C. E., Vol. LXXIX (1915), page 238.

DISCUSSION

Mr. Church.

IRVING P. CHURCH,* ASSOC. AM. SOC. C. E. (by letter).—All the algebraic relations and formulas of this interesting and well-arranged paper have been verified by the writer. In the derivation of Formula (R), however, it would seem that a special assumption is necessary, which is not mentioned in the paper. Without this assumption, the writer's analysis leads to an expression differing from the author's in a numerical coefficient, simply, involving a discrepancy of about 10 per cent.

As of some possible interest, the writer herewith presents his analysis in the case of Formula (R).

As the calculus is to be used, it will be necessary to supplement and modify the author's notation somewhat, as follows:

Let t denote the time at any instant from the beginning of the (upward) motion of the water surface in the air tank;

Let T (and not t) denote the time for the pressure in the air tank to change from p_1 to p_2;

Let p be the air pressure (in feet of water), in the tank at any time; (and other notation as required).

The case of load thrown off, only, will be treated; air compressed. From Assumption 1, we have:

$$\frac{d\,p}{d\,t} = C' \text{ (a constant)}$$

and, therefore,

$$p = p_1 + C'\,t \qquad (1)$$

As Assumption 2 is not made, we have the relation,

$$A\,\frac{d\,y}{d\,t} = a\,(v - V_2) \qquad (2)$$

and, from Assumption 3 (that is, y is zero compared with p),

$$W a \,.\, \frac{L}{g} \,.\, \frac{d\,v}{d\,t} = -\,(p - p_1)\,W a \qquad (3)$$

Substituting in Equation (3) the value of p from Equation (1), there is obtained,

$$\frac{L}{g} \,.\, d\,v = -\,C'\,t\,d\,t \qquad (4)$$

in which the variables, v and t, are separated.

By integrating Equation (4) between the limits, v and V_1, for v and t, and zero for t, we find, after transposition and division by $\frac{L}{g}$,

$$v = V_1 - \frac{C'\,g}{L} \,.\, \frac{t^2}{2} \qquad (5)$$

which gives v as a function of t at any instant.

* Ithaca, N. Y.

Now, insert in Equation (2) the value of v from Equation (5), whence, Mr. Church.

$$A \cdot d\,y = a\,(V_1 - V_2) \cdot d\,t - \frac{C'\,g\,a}{2\,L} \cdot t^2\,d\,t \ldots\ldots\ldots(6)$$

where the variables y and t are separated.

The integration of Equation (6), with limits of y and zero for y, and t and zero for t, leads to,

$$y = \frac{a}{A}(V_1 - V_2) \cdot t - \frac{C'\,g\,a}{6\,A\,L} \cdot t^3 \ldots\ldots\ldots\ldots(7)$$

that is, y as a function of t.

We are now ready to find a relation between Y (that is, maximum y) and the corresponding time, T, as follows:

Substituting Y for y, and T for t, in Equation (7), we have, after re-arranging the terms,

$$Y = \left(V_1 - V_2 - \frac{C'\,g\,T^2}{2\,L}\right)\frac{a\,T}{A} + \frac{C'\,a\,g\,T^3}{3\,A\,L} \ldots\ldots\ldots(8)$$

With V_2 for v, and T for t, in Equation (5), however, we find,

$$V_1 - V_2 - \frac{C'\,g\,T^2}{2\,L} = \text{zero} \ldots\ldots\ldots\ldots\ldots\ldots(9)$$

and, consequently, Equation (8) becomes,

$$Y = \frac{C'\,a\,g\,T^3}{3\,A\,L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(10)$$

The constant, C', is now obtained by substituting p_2 for p, and T for t, in Equation (1), that is, $C' = \frac{p_2 - p_1}{T}$, which, in Equation (10), gives, finally,

$$Y = \frac{a\,g\,(p_2 - p_1)\,T^2}{3\,A\,L} \ldots\ldots\ldots\ldots\ldots\ldots(11)$$

that is,

$$T = \sqrt{3}\,\sqrt{\frac{L\,A\,Y}{a\,g\,(p_2 - p_1)}} \ldots\ldots\ldots\ldots(R')$$

instead of the author's Formula (R), the latter differing from Formula (R'), however, only in having $\frac{\pi}{2}$ instead of $\sqrt{3}$; that is 1.5708 instead of 1.732, a discrepancy of about 10 per cent.

The writer was then led to seek the derivation of Formula (R) on other lines, by noting that the factor, $\frac{\pi}{2}$, was suggestive of a harmonic motion; and, accordingly, he started again with the assump-

Mr. Church. tion* that the motion of the water surface in the tank is harmonic, the initial point of this motion being the mid-point of a harmonic oscillation, so that the time from the start to the highest point would be,

$$T = \frac{\pi}{2}\sqrt{\frac{1}{C''}} \qquad (12)$$

where C'' is the constant, the product of which by the distance (this product being taken with a negative sign), gives the acceleration, a', of the motion at any instant, namely,

$$a' = -C''\,y \qquad (13)$$

Now $a' = \frac{d^2 y}{d\,t^2}$, an expression for which may be obtained by differentiating the two members of Equation (2) with respect to t; whence,

$$A \cdot \frac{d^2 y}{d\,t^2} = a \cdot \frac{d\,v}{d\,t} \qquad (14)$$

in which, if the value of $\frac{d\,v}{dt}$ from Equation (3) be inserted, there results,

$$a' = -\frac{a\,g}{A\,L}(p - p_1) \qquad (15)$$

or, for the highest point,

$$a'_2 = -\frac{a\,g}{A\,L}(p_2 - p_1) \qquad (16)$$

From Equation (13), however, we also have for the highest point,

$$a'_2 = -C''\,Y \qquad (17)$$

and the equating of these two values gives,

$$C'' = \frac{a\,g}{A\,L} \cdot \frac{(p_2 - p_1)}{Y} \qquad (18)$$

the insertion of which in the expression for T in Equation (12) leads to the author's Formula (R), namely,

$$T = \frac{\pi}{2}\sqrt{\frac{L\,A\,Y}{a\,g\,(p_2 - p_1)}} \qquad (R)$$

It is to be noted that the "harmonic" assumption just used, to take the place of the author's Assumption 1, simply consists in making $p - p_1$ proportional to the distance, y, instead of to the time, t; that is, in assuming the pressure in the tank to rise at a constant distance-rate, instead of at a constant time-rate. This is indicated by the

* This takes the place of Assumption 1.

fact that, from Equation (15), $p - p_1$ is proportional to the acceleration, a', of the motion of the surface of the water in the tank, and that, by Equation (13), this acceleration is assumed to be proportional to the distance, y (and of contrary sign), which is a characteristic property of harmonic motion. Mr. Church.

R. D. JOHNSON,* ESQ. (by letter).—Mr. Warren has touched on the fact that such formulas for the simple air tank as those proposed by him have only a limited practical application, because they furnish no index of the nature of the surge wave, as to whether or not it will die out of its own accord after its inception. This is a general fault with formulas which neglect governor action. Mr. Johnson.

Where differential action is omitted, there is much likelihood of falling into the error of designing the tank too small, and inflicting on the power plant a pressure accumulator instead of providing it with a regulator, and thus causing far more trouble than that which is intended to be cured.

Therefore, the writer has thought it desirable to supplement his equations with the following analysis, which serves to fix a minimum limit to the size of the simple air tank, independent of any formula which purports to give the value of the first dip of the pressure wave, following a load change.

If a surge wave is an increasing one, no perceptible initial load change is required to quicken it into life, because the system is already in a state of unstable equilibrium. Therefore, if, by studying the effect of very small load changes, an accurate solution of the critical size of a tank can be obtained, there is perfect assurance that, if the wave once begins to augment, it will continue to do so indefinitely.

On the other hand, if the study of the infinitesimals shows a tendency of the wave to die out, it seems reasonable that all appreciable surges would show the same thing.

It is necessary to regard only very small load changes in applying this theory, because, in that case, certain quantities are negligible which otherwise would complicate the mathematics beyond the possibility of manipulation.

In the following, let Z be the total dip of the pressure wave, at the end of the first quarter cycle, from its initial quiescent level, or, in Mr. Warren's nomenclature, $Z = Y + p_1 - p_2$, and let z be the variable, Z.

The following well-known fundamental equations hold good when governor action and friction are neglected, and V_2 is regarded as a constant draft velocity of the water-wheel.

$$d\,t = \frac{\left(\frac{L}{g}\right) d\,v}{z}, \text{ and } d\,y = \frac{(V_2 - v)\,a\,d\,t}{A}.$$

* New York City.

Mr. Johnson. Also, when Y is small compared to l, we may write,

$$d\,z = \frac{p_1 + l}{l}\,d\,y.$$

Eliminating dt, we have,

$$\int_0^Z z\,d\,z = \frac{a\,L\,(p_1 + l)}{A\,l\,g}\int_{V_1}^{V_2}(V_2 - v)\,dv.$$

From which,

$$Z = \sqrt{\frac{a\,L\,(p_1 + l)}{A\,l\,g}}\,(V_2 - V_1)\ldots\ldots\ldots\ldots\ldots(1)$$

This expression holds good only for a small value of Z when V_2 is very little larger than V_1, the difference being called Δv.

The wave described by this formula would live everlastingly at its initial amplitude, that is, when not influenced either by the damping action of friction or the accelerating effect of governor action, and all dimensions of air tanks would fulfill the so-called critical condition, of a uniform, everlasting wave.

It follows, logically, that if the damping action of friction is just equal, in its effect, to the augmenting action of the governor, then the wave continues in the same condition as though neither influence was present.*

Therefore, if these two opposing factors are equated, the critical dimensions of the air tank become apparent.

The increment to Δv caused by the effort of the governor to maintain constant power as the pressure drops, is, naturally, $\dfrac{Z\,V_2}{h - Z}$, where h is the net head corresponding to p_1; or, as Z is here considered insignificant as compared to h, the increment to Δv may be written

$$\frac{Z\,V_2}{h}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

The corresponding increment to Z or Δz may be found, by reference to Equation (1), to be,

$$\Delta z = \sqrt{\frac{a\,L\,(p_1 + l)}{A\,l\,g}} \times \frac{Z\,V_2}{h}\ldots\ldots\ldots\ldots\ldots(3)$$

Now, the opposing change in friction head, due to Δv, may be obtained from the differential expression, $c\,((V_2 + \Delta v)^2 - V_2^2)$, which becomes (neglecting $\overline{\Delta v^2}$ as an infinitesimal of the second order)

$$\Delta f = 2\,c\,V_2\Delta v\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

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* See "The Surge-Tank in Water-Power Plants", *Transactions*, Am. Soc. Mech. Engrs., 1908, p. 840.

Mr. Johnson.

Or, substituting the value of Δv from Equation (1),

$$\Delta f = 2\, c\, V_2\, Z \sqrt{\frac{A\, l\, g}{a\, L\, (p_1 + l)}} \quad \ldots\ldots\ldots\ldots\ldots (5)$$

Now, when $\Delta z = \Delta f$, the condition of instability sought is realized.

Equating their values, as derived in Equations (3) and (5), we have, for the dimensions of the critical size of a simple air tank,

$$\frac{A\, l}{p_1 + l} = \frac{a\, L}{2\, g\, c\, h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)^*$$

If A or l is smaller, or p_1 larger, than in the combination which equals this expression, then the design becomes, theoretically, an impossible one, and practically, also, unless some existing condition, such as partial differential action, a sluggish governor, or additional friction of some sort, has been overlooked.

Equation (6) involves the idea of constant efficiency of water-wheel, or a horizontal line expressing the relation between power output and efficiency, and therefore holds good only at the crest point of the efficiency curve through an infinitesimal range of surging.

If the water-wheel is operating on the drooping side of the efficiency curve, the tank dimensions need to be still more ample to reach the critical value.

If the negative tangent of the efficiency-power curve $\left(\text{that is, } \frac{\Delta E}{\Delta P}\right)$, where E is the efficiency and P is the power, be called tan. α, the correct expression involving this new idea is,

$$\frac{A\, l}{p_1 + l} = \frac{a\, L\, (E + \frac{3}{2}\, P \text{ tan. } \alpha)}{2\, g\, c\, h\, E} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

A numerical substitution in Equation (6) indicates that Mr. Warren's chosen example falls far below the critical size, and, in such cases, as he has elsewhere intimated, his value of p_2 cannot be said to constitute even an approximation to the true magnitude of the maximum or minimum air pressure.

It must not be understood that Equation (7) provides a sufficiently large tank for practical purposes, because it distinctly does not.

An everlasting wave is a constant invitation to trouble, due to partial synchronous conditions between the period of load-changes and that of the pressure wave, and, ordinarily, a commercially practicable simple air tank would need to be much larger than indicated by Equation (7).

It follows, therefore, that even correct equations, impossible to derive, relating to the first quarter cycle of the wave in a simple air

* When l becomes infinite, this equation is seen to agree with the Thoma formula for the open simple tank.

Mr. Johnson. tank which is unmodified by adequate choking devices, would be of very doubtful practical value, because the cost of a commercially operative simple air tank is usually found, from Equation (7), to be prohibitive.

Mr. Wahlman. P. WAHLMAN,* M. AM. SOC. C. E. (by letter).—For several years the writer has been interested in surge-tank regulation, but, so far, has not found any reliable formulas for the simple tank (that is, without the differential arrangement), whether open or closed as an air tank. Consequently, he has taken interest in testing the practical value of Mr. Warren's new formulas with the help of the tedious but accurate process of arithmetic integration.

For this test the author's own tank (page 261) has been selected, and, in the arithmetical work, the friction as well as the governor action has been considered, as neither can be neglected in practice. As the friction and entry loss in the 2 100-ft. conduit are not given, it has been necessary to assume a value of that combined loss, such as $F = 0.07\ V^2$. The static head at the wheels has been assumed to be 459 ft.

Fig. 2 shows graphically the result of a complete shut-down from full load, or from $V = 12.5$. In this case, the governor has no influence, of course, and only the friction makes this practical case different from the author's first case. The correct result is $Y = 20.64$ and $p_2 =$ 779, as compared with the author's 20.0 and 764, or 20.8 and 783 for isothermal expansion; and, consequently, it may be assumed that Equations (A) and (B) are practically correct for this particular case.

When the load is thrown on, the governor action is added and cannot be disregarded. The result of full load thrown on is shown graphically in Fig. 3, and a glance at these curves shows that, due to the governor, the drop in head and the simultaneous increase in draft velocity, V_d, is so rapid, that the conduit velocity, V_c, cannot be accelerated fast enough to catch up with the draft velocity, which means that the full load cannot be maintained, even if the wheels were large enough to take the steadily increasing flow and their efficiency was constant. It is evident, therefore, that, for full load thrown on, the author's formulas are impractical.

Fig. 4 demonstrates the conditions when half load is thrown on suddenly. When the conduit velocity has first caught up with the draft velocity, Y is only 16.87, and $p_2 = 354$, and if the arithmetic integration was stopped at this point, one might be inclined to believe that the tank in question was sufficiently large for half load thrown on; but, if the curves are worked out farther, their amplitude is found to increase for each surge, and will finally go beyond possible limits; that is to say, the tank or conduit will either burst or the tank will be emptied.

* New York City.

Mr. Wahlman.

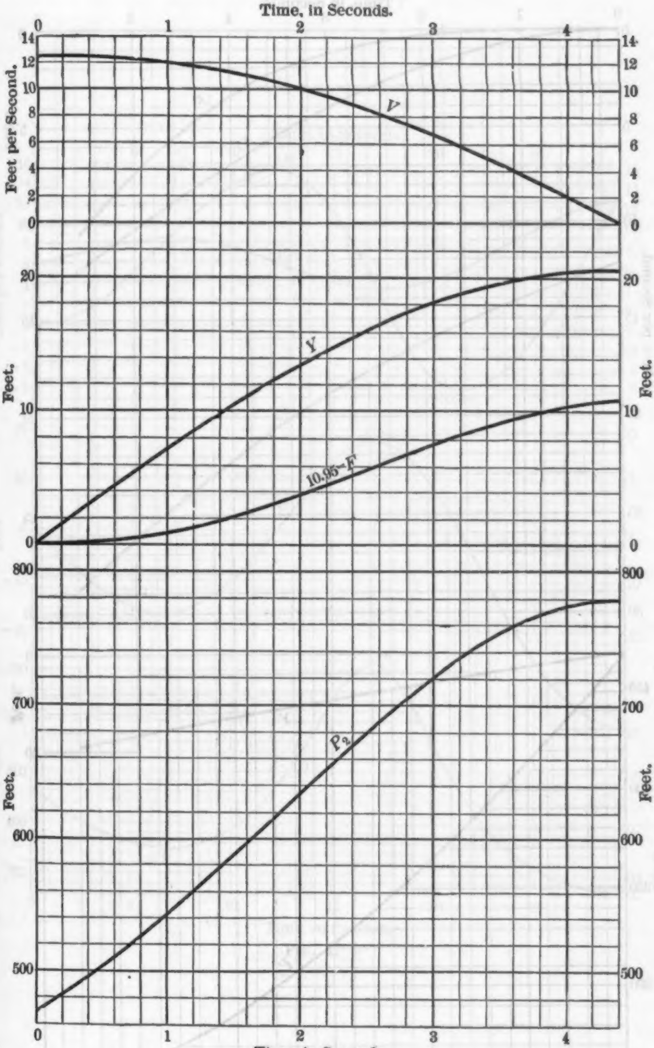

FIG. 2.

Mr. Wahlman.

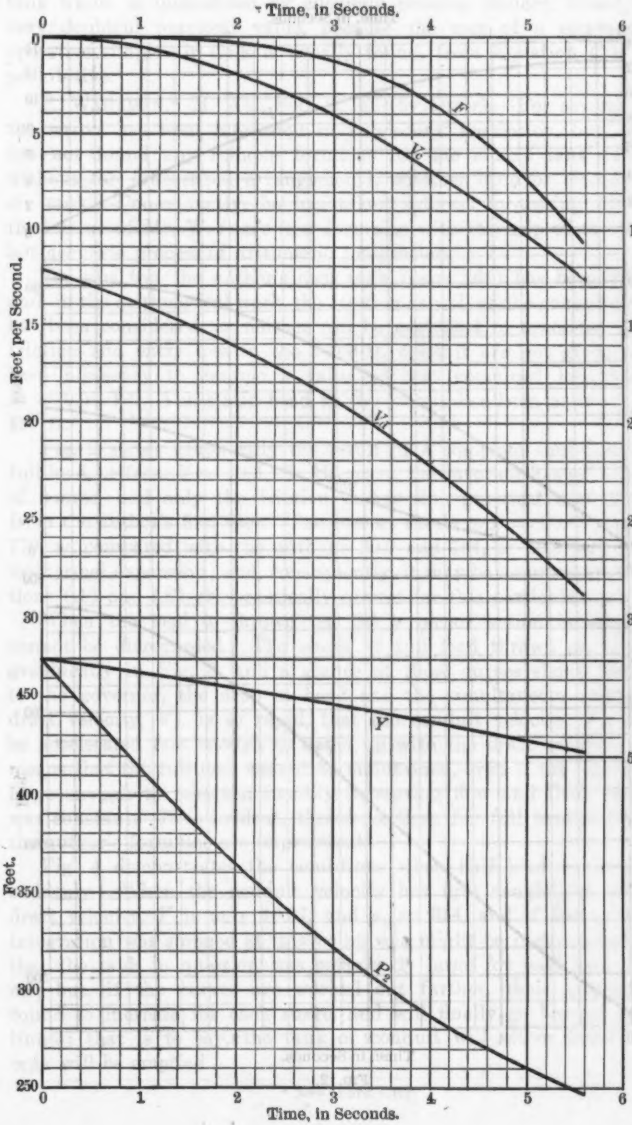

FIG. 3.

Mr. Wahlman.

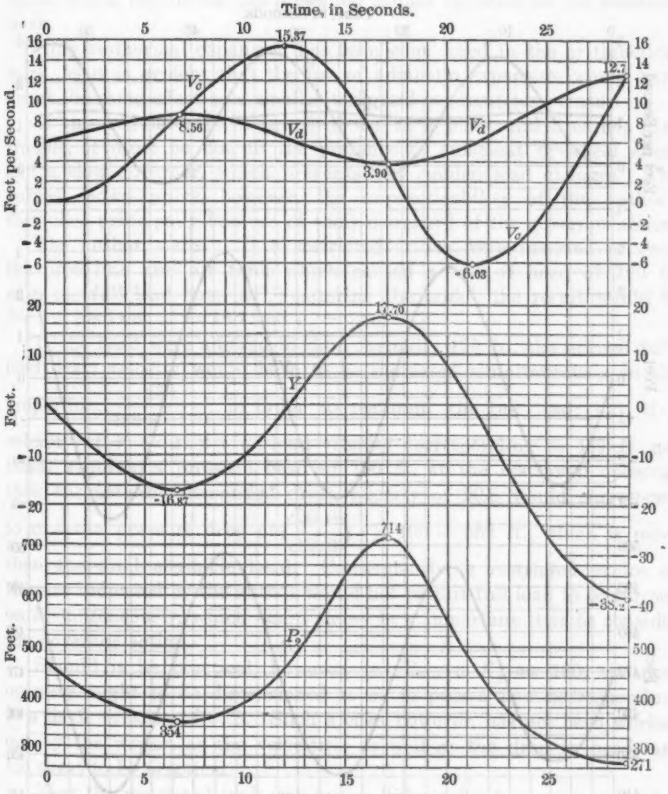

FIG. 4.

Mr. Wahlman.

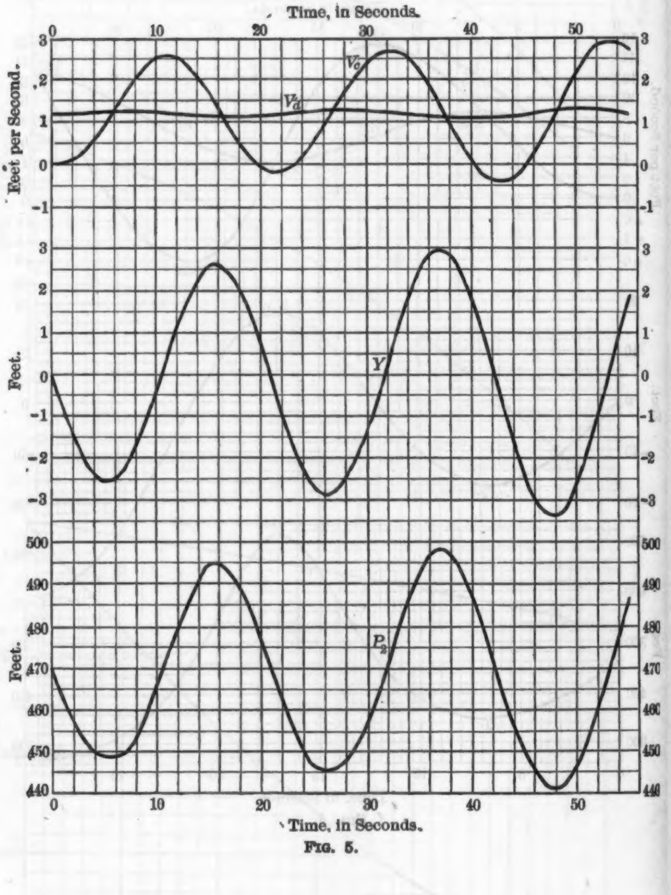

FIG. 5.

Mr. Wahlman.

Fig. 5 finally shows that even as small a sudden load change as 10% of full load will cause surges which will grow indefinitely. Consequently, it is evident that an air regulator of the dimensions indicated in the author's example is not only entirely inadequate for any sudden load changes, but even spoils regulation altogether, and that the formulas which the author has presented cannot be relied on for practical work.

Only isothermal expansion has been considered in the arithmetical work, as it is evident that the use of adiabatic expansion would have about the same effect as a smaller tank and constant temperature.

If this regulator is in practical use at a plant and does not give trouble, it must be due to the presence of sufficient fly-wheel effect and a slow governor, which prevents any sudden load changes. The curves shown prove, however, the impracticability of the author's formulas, principally because of their disregard of the governor action.

The author states: "If a restricted orifice were inserted between the pipe line and air tank which caused a loss of head of 190 ft. with the full load flow (aV_1) entering the tank", the result would be for full load on: $Y = 18.7$ and $p_2 = 346$.

If the head were allowed to drop instantly 190 ft., the normal full-load draft velocity would have to be increased simultaneously in the proportion, $\frac{459}{459 - 190}$, in order to maintain the full load; but, if a velocity of $V = 12.2$ (full-load velocity corresponding to 459 ft. net head) requires a pressure head of 190 ft. at the restricted opening, then the actually demanded draft velocity of 20.8 would correspond to an actual pressure drop of $\left(\frac{20.8}{12.2}\right)^2 \times 190 = 552$ ft., which is more than the total available head. Consequently, a restricted orifice of the size indicated by the author would not permit full load to be thrown on instantaneously, which again shows how important it is to consider the governor action.

Should it be practical, however, to allow a $K_1 = 190$, the port opening ought to be dimensioned so as to pass a flow corresponding to a draft velocity of 20.8. Such a case, however, has not been worked out by the writer, as the velocities, as well as the drop in head, are too great to be practical.

That the restricted tank opening or differential feature is a decided improvement on the simple tank, however, the writer has had occasion to ascertain, while working out several surge-tank problems, and this applies most forcibly in the case of the air tank.

A good example of a well designed air regulator appears in the Annual Report of New York State Water Supply Commission for 1909, on page 326.

Mr. Jorgensen.

L. R. JORGENSEN,* M. AM. SOC. C. E. (by letter).—This subject of air tanks on pipe lines is entitled to receive greater attention in the future than it apparently has in the past. There seems to be no reason why such installations should not be of advantage, especially where turbines are the prime movers in the power-house. Undoubtedly, there is a field for air tanks on modern pipe lines, as they act as safety valves in addition to whatever relief device may be provided for the turbines. Mr. Warren's paper is of much value to hydraulic engineers. On high-head plants where impulse wheels are used, the proposition is somewhat different, especially if the needle regulating the size of the jet is hand-operated and the governing is effected by deflecting the jet away from, or into, the wheel buckets. On such a pipe line, there would be less need for air tanks, as the changes in the velocity of the water in the pipe line are under hand control.

The writer was once connected with a high-head hydro-electric plant in Southern California where the static head was more than 1 900 ft. There were several air tanks along the pipe line, each tank connected to the line by a short piece of pipe and provided with a valve. Soon after being put into use, these tanks filled up with water completely, as no compressor arrangement had been provided to furnish the necessary air to compensate for air absorption and air leakage from the tanks. As it was also found, however, that the pipe line operated satisfactorily without the air tanks, no compressor was put in. Since then some of the air tanks have been removed and others have been simply disconnected. This plant is regulated by deflecting needle nozzles, the needles being operated by hand whenever great changes of load take place. Had the governor operated the needle direct—in connection with an auxiliary needle for relief—it is very probable that the air tanks would have been advantageous in regulating the pressure in the long pipe line. The erection of a compressor and air pipe system to make the tanks operate properly would not have been much of an undertaking.

A short time ago, the writer had occasion to inspect another high-head plant (1 800 ft.) in California, in which the needle inside the nozzle was operated by the governor direct—in connection with an auxiliary needle for relief. There were strong indications that an air tank on this line would have been very useful in preventing excess pressure, especially at times when the load was low and going still lower.

Mr. Warren.

MINTON M. WARREN,† ASSOC. M. AM. SOC. C. E. (by letter).—The writer was much gratified to find that his formulas for air tanks had been verified by Professor Church, and by actual data of Mr. Wahlman,

* San Francisco, Cal.

† Cambridge, Mass.

in the case where they were properly applied. He also wishes to thank Mr. Johnson and Mr. Jorgensen for their discussions. Mr. Warren.

The formulas and object of the paper have been somewhat misunderstood, as the formulas were offered merely as a guide to air tank design, and to be used only with the full realization of the assumptions on which they were based. Any engineer applying them must, of course, use his judgment in fitting them to the special case he is designing.

As far as the writer knows, no useable formula of any kind has been offered for air tank design, and these are presented as a start for such computations. As governor action, wheel efficiency, friction, etc., cannot be neglected, the results must be modified in each particular case, and differential devices, valves, or orifices, must be used in order to prevent surges and aid regulation.

Before publication the formulas were checked by the results of an actual operating air tank which had been very successful in removing trouble caused by surges and water-hammer in a large power plant.

These data are not at present available to the writer, but the results of actual tests of a new air tank design along these lines will soon be ready, and when published should give valuable information on this subject.

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Paper No. 1408

CONSTRUCTION PROBLEMS OF THE MANHATTAN-BRONX, AND LEXINGTON AVENUE SUBWAY JUNCTION AND QUEENSBOROUGH TUNNEL CONNECTIONS*

By George Perrine, M. Am. Soc. C. E.

With Discussion by Messrs. John Hays Myers, Clarence E. Carpenter, Robert A. Shailer, Robert Ridgway, Henry H. Quimby, and C. V. V. Powers.

Synopsis.

The object of this paper is to describe the principal construction problems that confronted the contractor in making the connection between the present subway in Park Avenue and the new Lexington Avenue line in the vicinity of 42d Street, Borough of Manhattan, New York City, and also in the extension of the Queensborough Tunnel.

The greater part of this contract is made up of special construction. As the standard construction in the New York subways has often been described, it will not be dealt with in this paper.

General Description.

On September 11th, 1914, the Public Service Commission for the First District of New York invited proposals for the construction of Section No. 1 of Route 43, a part of the Seventh Avenue Rapid Transit Railroad, and Section No. 1 of Route 26, a part of the Steinway Tunnel, now called the Queensborough Tunnel. The Commis-

* Presented at the meeting of December 19th, 1917.

sion included in this proposal a spur to the existing Manhattan-Bronx Rapid Transit Railroad.

The limits of the routes covered by this paper are as follows: Section No. 1 of Route 43 (Fig. 1) extends from a point about 50 ft. south of the south side of 38th Street to the north side of 42d Street. It also includes the spur-track connecting with the present subway at Vanderbilt Avenue, and the ramp from the Queensborough Elevator Shaft to the Grand Central Station. Section No. 1 of Route 26 is in Forty-second Street, below the Diagonal Station, and extends from Vanderbilt Avenue to a point about 100 ft. east of the east building line of Lexington Avenue.

The contract was executed on December 4th, 1914, and the time allowed for the completion of the work was 28 months.

TABLE 1.—PRINCIPAL QUANTITIES.

	Section 1, Route 43.	Section 1, Route 26.
Earth excavation, in cubic yards.....	65 000	300
Rock excavation, " " ".....	45 000	130
Tunnel excavation, " " "	25 500	24 900
Steel, in tons.....................	4 725	309
Concrete, in cubic yards..............	26 000	11 850

The general plan of construction called for a sub-surface railroad having four tracks in the case of Route 43, and sub-surface railroads having two tracks in the cases of Route 26 and the spur.

The work under the contract included the care and support of buildings, sewers, pipes, railroads, with their rolling stock, and other surface, sub-surface, and overhead structures; the maintenance of traffic; the safety and protection of passengers and other persons; the restoration of pavements and other surfaces; and the removal and reconstruction of portions of the Manhattan-Bronx Rapid Transit Railroad and portions of the existing Queensborough Tunnel, in order to provide a connection with the new work.

The contract also provided that the adjacent portions of the Queensborough Tunnel be put into operation before the completion of the work under contract, and that the work be conducted in such a manner as not to interfere with or interrupt the safe and continuous operation of the trains, and avoid injury to passengers or other persons.

Shafts for the Disposal of Excavated Materials.—The contract required that the work on Section 1, Route 43, be prosecuted from within the limits of the property acquired by the City, bounded by

the east line of Park Avenue, the north line of East 41st Street, and the south line of East 42d Street. This was formerly the site of the Grand Union Hotel, the buildings on which had been torn down and removed by other contractors.

All material was taken into or removed from the property through a driveway across the north sidewalk of East 41st Street.

Excavated material from the Queensborough Tunnel level was taken up through the shaft east of First Avenue after it had been transmitted through the north tube—which the contract provided could be used for construction purposes—and also up through the ventilating shaft on Park Avenue, near the southeast corner of Park Avenue and East 42d Street.

The contractor was permitted to have an additional shaft in Park Avenue adjacent to the east side of the parkway at East 40th Street. This shaft will be back-filled after the completion of the work.

Excavation.—The excavation for the Diagonal Station crossing the lot east of Park Avenue, between 41st and 42d Streets, was done entirely as open cut. The material, both earth and rock, was removed by derricks and locomotive crane. The portion of the work in Park Avenue, between 42d and 40th Streets, which consisted in reconstructing the present tunnel for north-bound trains, and the section in 42d Street, was done by the usual cut-and-cover method. The south-bound local and the express tracks, running south from the diagonal station, were completed by tunneling. The Queensborough Tunnel, which was originally a two-track road, each track in a separate tunnel, was widened by the slice method, and thus the two single-track tunnels were changed into a single-span arch, forming the new Grand Central Station.

Sewers.—The sewers to the west of Park Avenue were led into a large chamber built just west of the new south-bound local tunnel.

Many changes were made necessary in the sewer system on account of the construction of these sections. The most important part of the sewer reconstruction was that running from Park Avenue and 41st Street. It consisted of three 42-in. cast-iron pipes, under all the subways, and an 8 by 8-ft. concrete sewer to the East River. The contractor built this section to a point 200 ft. east of Third Avenue.

Duct Lines.—The duct banks of the present subway, from 33d to 42d Street, had been laid between the tracks in the north-bound

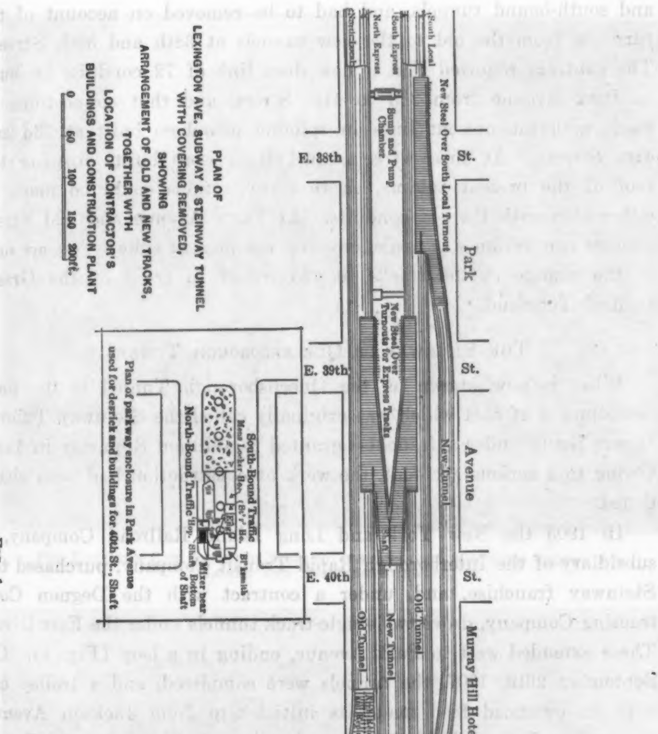

PLAN OF
LEXINGTON AVE. SUBWAY & STEINWAY TUNNEL
WITH COVERING REMOVED,
SHOWING
ARRANGEMENT OF OLD AND NEW TRACKS,
TOGETHER WITH
LOCATION OF CONTRACTOR'S
BUILDINGS AND CONSTRUCTION PLANT
0 50 100 150 200ft.
Plan of parkway enclosure in Park Avenue
used for derrick and buildings for 40th St., Shaft
South-Bound Traffic
North-Bound Traffic
Mens Locker Ho.
Mixer near Bottom of Shaft
E. 38th St.
E. 39th St.
E. 40th St.
E. 41st St.
E. 42nd St.
Park Avenue
Murray Hill Hotel
Hotel Belmont
Grand Central Terminal
Vanderbilt Ave.
Lexington Avenue
South Local
South Express
North Express
North Local
New Steel Over South Local Turnout
Sump and Pump Chamber
New Steel Over Turnouts for Express Tracks
New Tunnel
Old Tunnel
Shaft
6' x 6' Sewer
New 8' x 8' Concrete Sewer
Elec. Manholes of Duct Line From 33d Street
Vent'r Shaft of Steinway Tunnel Used as Work Shaft
Stone & Sand Bins under walk
New Platform for Shuttle Trains to and from Broadway
Ramp to G. C. Terminal
Concrete and Tower Chute
Platform
Steinway Station Platform
Foremen & Store Ho.
Contr's Office
Carpenter Shop
Blacksm.
Dr's Off.
Terry & Tench
10" Air Pipe from Power Plant at 42d St. and East River
15-Ton Locomotive Crane used on Station Excavation in Addition to Derricks
The Principal part of Rock from Steinway Tunnel was removed through Shaft at East River. Concrete was mixed at the River Shaft and distributed in Cars drawn by Electric Trolley Locomotive.

FIG. 1.

and south-bound tunnels, and had to be removed on account of the turnouts from the old to the new tunnels at 38th and 39th Streets. The contract required that a new duct line of 72 conduits be built in Park Avenue from 33d to 41st Street, and that connections be made with the present lines in splicing chambers built at 33d and 41st Streets. At these streets, duct lines were built, crossing the roof of the present subway, so that connections could be made at either side with the old conduits. At Fifth Avenue and 42d Street another connection was made, crossing the present subway, on account of the change in the tracks in 42d Street in front of the Grand Central Terminal.

The Steinway or Queensborough Tunnel.

What is now known as the Queensborough Tunnel is the final development of that which was originally called the Steinway Tunnel. It was begun under a franchise granted to William Steinway in 1890. Owing to a serious accident, the work of construction had been abandoned.

In 1905 the New York and Long Island Railroad Company, a subsidiary of the Interborough Rapid Transit Company, purchased the Steinway franchise, and, under a contract with the Degnon Contracting Company, drove two single-track tunnels under the East River. These extended west to Park Avenue, ending in a loop (Fig. 4). On September 26th, 1907, the tunnels were completed, and a trolley car with an overhead feed made its initial trip from Jackson Avenue Station, in Long Island City, to what is now known as the Grand Central Station of the Queensborough Tunnel, just east of Lexington Avenue.

During construction the tunnel was usually spoken of as the Belmont Tunnel, August Belmont being at the time President of the Interborough Transit Company.

Owing to litigation over the alleged expiration of the franchise, the running of the car was discontinued, and the tunnel remained unused until October, 1907, when the car was removed from the tunnel.

In 1913 the tunnels were purchased by the City and incorporated as a section of the Dual Subway System. The work of reconstruction from an overhead trolley system to one operated by the third rail was commenced. The tunnels were opened to public traffic on June 22d,

FIG. 2.—OPEN CUT, LOOKING TOWARD 41ST STREET, SHOWING DRIFTS FOR LOCAL AND EXPRESS TUNNELS.

FIG. 3.—GREAT ARCH COMPLETED. STATION PLATFORM ADJOINING EAST SIDE OF ELEVATOR SHAFT.

Fig. 2.—Open Cut [illegible]

Fig. 3.—[illegible] Station [illegible] Elevator [illegible]

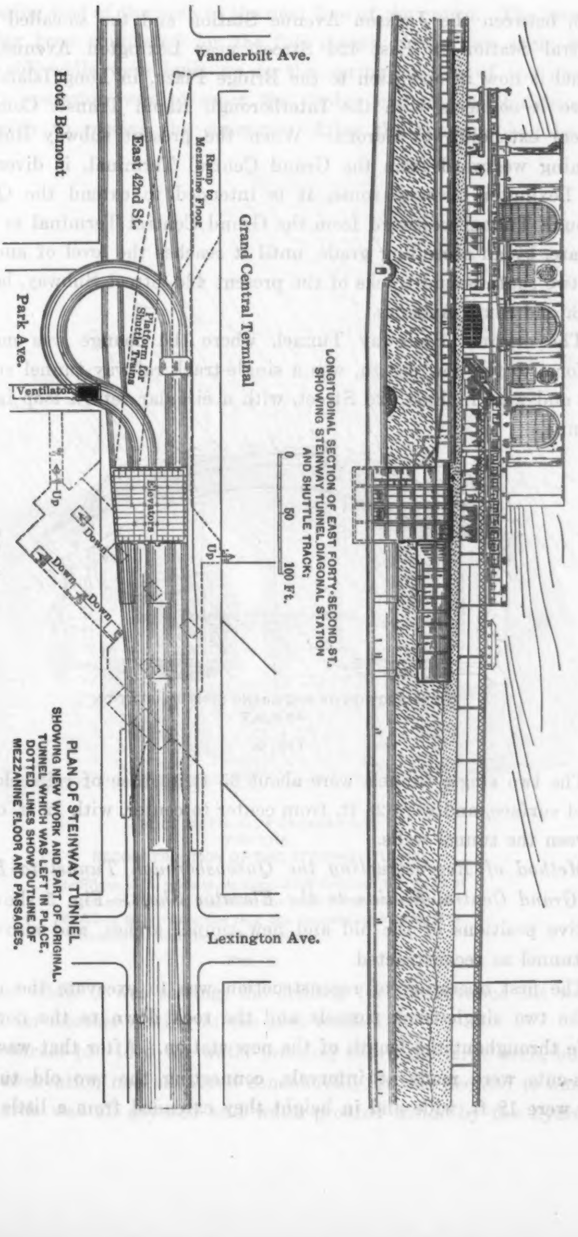
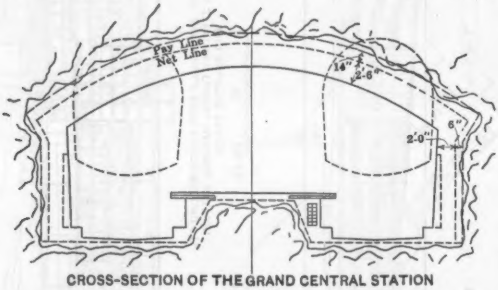

Vanderbilt Ave.
Grand Central Terminal
Ramp to Mezzanine Floor
East 42nd St.
Hotel Belmont
Platform for Shuttle Trains
Park Ave.
Ventilator
Elevators
Up
Down
Down
Down
Down
Up
LONGITUDINAL SECTION OF EAST FORTY-SECOND ST. SHOWING STEINWAY TUNNEL DIAGONAL STATION AND SHUTTLE TRACKS
0 50 100 Ft.
Lexington Ave.
PLAN OF STEINWAY TUNNEL
SHOWING NEW WORK AND PART OF ORIGINAL TUNNEL WHICH WAS LEFT IN PLACE.
DOTTED LINES SHOW OUTLINE OF MEZZANINE FLOOR AND PASSAGES.

FIG. 4.

1915, between the Jackson Avenue Station and the so-called Grand Central Station in East 42d Street, near Lexington Avenue. The tunnel is now in operation to the Bridge Plaza, in Long Island City, where it connects with the Interborough Rapid Transit Company's system extending to Corona. When the present subway line, now running westward from the Grand Central Terminal, is diverted to the Lexington Avenue route, it is intended to extend the Queensborough Tunnel westward from the Grand Central Terminal to Times Square, by an ascending grade, until it reaches the level of and joins the two south-bound tracks of the present 42d Street Subway, between Fifth and Sixth Avenues.

The original Steinway Tunnel, where the change was made to conform to the new design, was a single-track railway tunnel running east and west in East 42d Street, with a circular return loop in Park Avenue.



CROSS-SECTION OF THE GRAND CENTRAL STATION AS BUILT

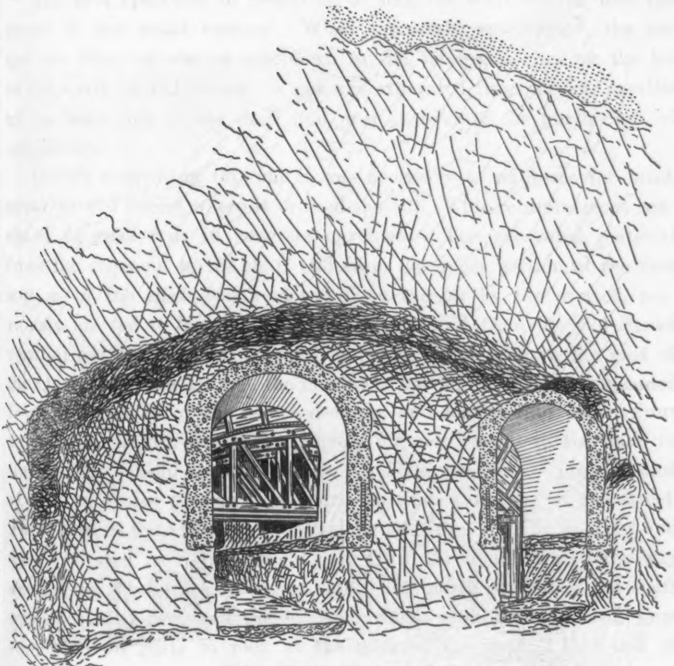
FIG. 5.

The two single tunnels were about 65 ft. to base of rail below the street surface, and were 28 ft. from center to center, with 12 ft. of rock between the tunnel walls.

Method of Reconstructing the Queensborough Tunnel to Extend the Grand Central Station to the Elevator Shaft.—Fig. 5 shows the relative positions of the old and new tunnel arches, and also shows the tunnel as reconstructed.

The first operation of reconstruction was to excavate the inverts of the two single-track tunnels and the rock down to the new subgrade throughout the length of the new station. After that was done, cross-cuts were made at intervals, connecting the two old tunnels; they were 12 ft. wide and in height they extended from a little below

the springing line of the arch to the neat line of the crown. The cross-cut having been completed for the full extent of the new arch, the concrete side-walls were built up to the springing line of the arch. The next operation was to erect the timber centers and bulkheads above, ready to receive the concrete. After the concrete had been



SLICE METHOD OF EXCAVATING
FOR THE
RECONSTRUCTION OF THE STEINWAY TUNNEL

1.-Slices 15 ft. wide were excavated through the walls of the old tunnels, and then the arch form was erected.
2.-After concreting the ring the material adjoining it is excavated and the arch is built.

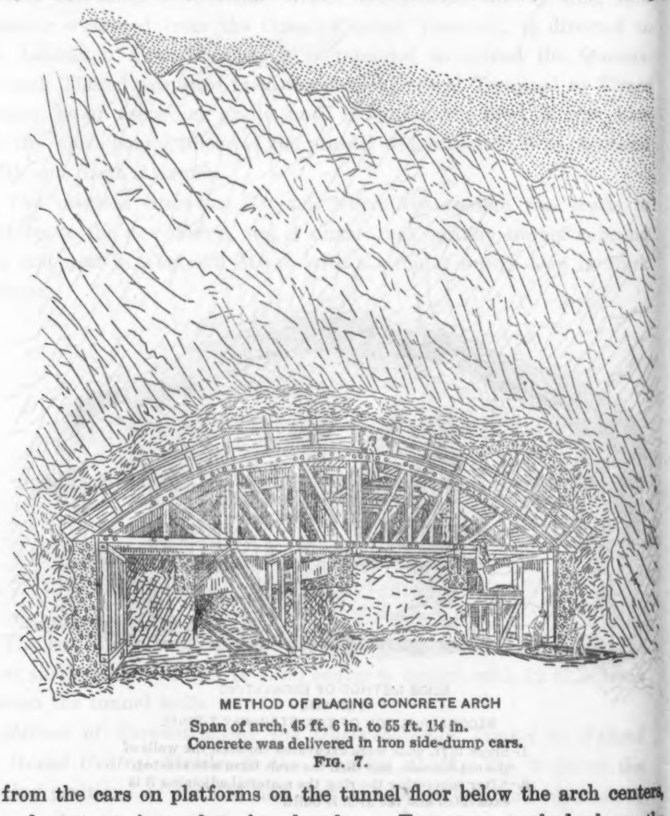
FIG. 6.

placed, and while it was setting, the drilling for the advance cut was completed for another setting of the arch centers.

The contract provided for water-proofing above the large arch, but, after the excavation had advanced considerably, it was decided to omit it. The arch was all grouted and water-proofed inside by the hydro-

lithic cement plaster process. The only timber used during the progress of the excavation for the large arch was that in the concrete arch center.

The concrete for the great arch was placed by hand. It was mixed at Shaft 2, at the East River, and hauled from the mixer to the work by an electric trolley through the north tube. The concrete was dumped



METHOD OF PLACING CONCRETE ARCH

Span of arch, 45 ft. 6 in. to 55 ft. 1¼ in.
Concrete was delivered in iron side-dump cars.
FIG. 7.

from the cars on platforms on the tunnel floor below the arch centers, and cast up into place by shovelers. Two men worked above the lagging, placing and tamping the concrete of the arch. A gang of fourteen men, working 24 hours, in 8-hour shifts, could fill one arch ring, comprising 100 cu. yd. of concrete.

Figs. 6 and 7 show the method of procedure in excavation and in concrete placing.

Queensborough Tunnel Elevator Shaft.—Another feature of the Queensborough Tunnel construction that was as important as the great arch was the elevator shaft. This shaft is about 63 ft. square to the neat line of the excavation, and is 71 ft. in depth to the sub-grade of the track.

The first operation in preparing to sink the shaft was to deck the street in the usual manner. When the shaft was started, the cut for the diagonal station was down to the sub-grade crossing the lot to the south of 42d Street. A cut was excavated from the lot, parallel to the west side of the shaft and near it, over to the north curb of 42d Street.

Before excavating this cut it was necessary to reinforce the brick sewer in 42d Street where it crossed the cut. This reinforcement consisted of steel rods in concrete envelopes. The excavated material from the cross-cut to the shaft and from the upper portion of the first section of the shaft was removed from the lot by the derrick, previously erected to handle the excavated material from the 42d Street ventilating shaft. Until it was possible to break through the roof of the Queensborough Tunnel, the muck was hoisted in buckets, suspended from the girders of the street decking, and loaded into cars. Four drifts were dug from the 42d Street lot, northward, to the curb in front of the Grand Central Terminal. These drifts were proportioned to receive lattice girders to support the timber decking of the street. These girders were from 40 to 44 ft. long and from 42 to 52 in. deep, and were procured from the Second and Third Avenue Elevated Railway during the three-track reconstruction. The north portion of the shaft was excavated first and the lattice girders, after being placed, were supported on posts to rock at the edge of the shaft. One end of each girder was near the north curb of 42d Street, the other end was on the center line of the elevator pit.

After the decking girders were in place, the excavation for the full north half of the shaft was begun. Several pipe lines, including a 30-in. water pipe, crossed the north half of the shaft excavation; also, while the sinking of the north portion of the shaft was in progress, the sewer crossing the south portion was by-passed in a 40-in. steel pipe.

When the mezzanine floor was completed, the decking girders for spanning the south half of the shaft were in place, ready to take the load of the street and the sidewalk from the temporary posting.

Two 52-ft. girders were placed at the same time as the decking girders for supporting those columns of the Elevated Railroad that came within the limit of the south part of the shaft excavation. The north ends of the decking and underpinning girders were all supported on the steel of the mezzanine floor, and this had to be reinforced in order to carry the loads properly distributed.

In connection with the Queensborough Tunnel there is a ventilating shaft near the southeast corner of 42d Street and Park Avenue. This shaft was joined with the south tube of the double tunnel section which runs west from the elevator shaft. These two single-track tunnels were included in the contract as far as Vanderbilt Avenue. The south tunnel passed the corner pier of the Hotel Belmont in solid rock, with only 10 ft. of rock between the concrete footing of the pier and the limit of excavation. The load on the footing was 850 tons. The two north-bound tracks of the Manhattan-Bronx Railway will be utilized for a shuttle train service between Broadway and the Queensborough Elevated Shaft. The extension of these tracks between Vanderbilt Avenue and the Elevator Shaft, and the ramp connecting the Elevator Shaft and the Grand Central Terminal are parts of the contract.

Route 43.

The principal features of interest on Route 43 are as follows:

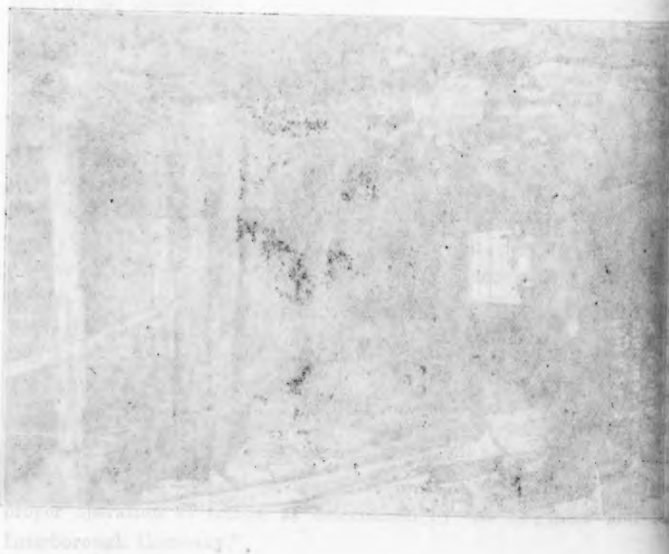
(1) The design and construction of the "half-arch" section over the express tracks and the south-bound local near 39th and 38th Streets, respectively; (2) lowering the invert of the north-bound local track and maintaining traffic; (3) constructing the south-bound local and the express tracks beneath the old tunnels, and erecting steel at the latter places.

The Half-Arch Construction.—This construction was suggested by the contractor's engineers after the contract had been let. The original design is shown on Plate II and Figs. 12 and 13. The specification stated that the contractor would be "required to allow and arrange with the Interborough Company to put in such cross-overs as may be needed between the existing tracks for the prosecution of the work. * * * To reconstruct or install and maintain such signals, lighting, and telephone cables as may be necessary for the proper operation of trains, as determined by the Engineer and the Interborough Company."

FIG. 8.—EXCAVATION FOR ELEVATOR SHAFT, LOOKING WEST.

FIG. 9.—NEW STATION ARCH OF QUEENSBORO TUNNEL, LOOKING WEST.

... the ... of the Grand Central Terminal ...



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Interborough Company".

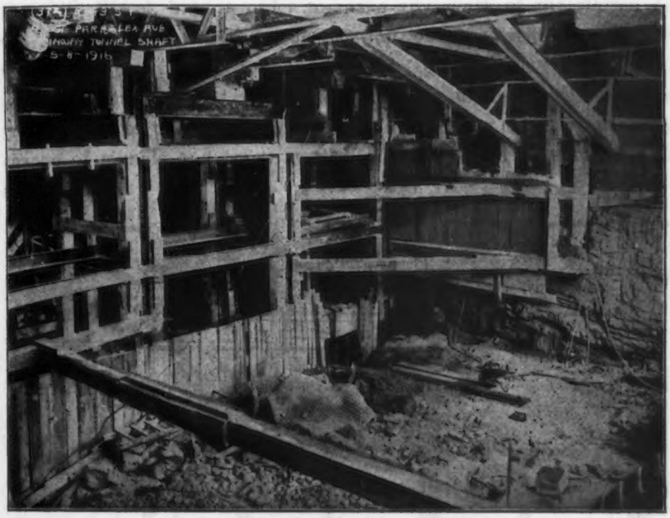


FIG. 10.—EXCAVATION FOR SOUTH SIDE OF ELEVATOR SHAFT COMPLETED TO GRADE OF ENGINE-ROOM FLOOR.

FIG. 11.—REAR OF SIDE DRIFT, SHOWING OLD ARCH DRILLED. BLOCKING IS SHOWN UNDER ARCH RESTING ON SHIELD.

Cross-overs could have been made south of the new construction near 38th Street, but, in order to make any south of the station at 42d Street, an elaborate reconstruction of the existing structure would have been required.

On account of the cross-overs and their maintenance, and the great danger to the traveling public in the Subway due to temporary change of signals and track system, it was decided to eliminate all track changes.

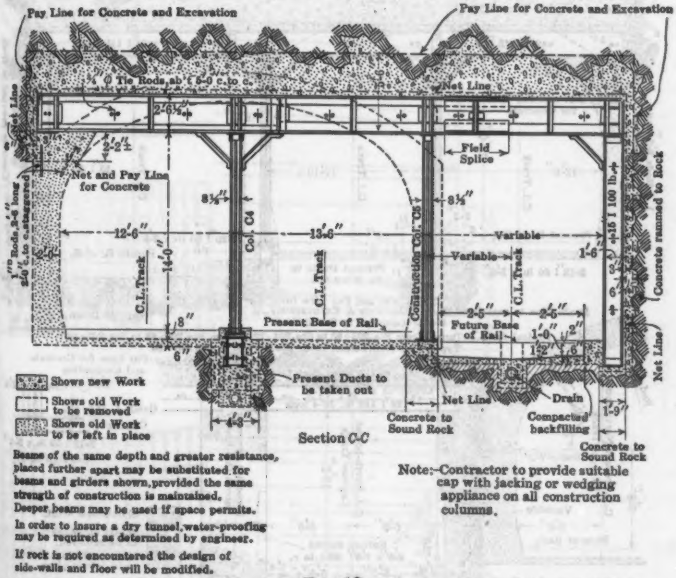


FIG. 12.

Drawings made in the contractor's field office, Figs. 14, 15, 16, and 17, show the several steps in the procedure for the proposed work. These plans contemplated removing one-half of the existing arch in short sections and erecting the new structure completely before removing another portion of the arch. The function of the new structure was to resist the crown thrust of the old arch and to support the rock above the new track. This plan omitted entirely any new work above the opposite tracks. In order to protect the trains and the tracks from any small pieces of falling material, a shield was designed for the

interior of the old tunnel and was erected before the work, above described, was commenced.

It was not claimed by the contractor, however, that the shield was designed to support large masses of falling concrete; it was built only to hold the concrete in place by fitting the ribs and lagging

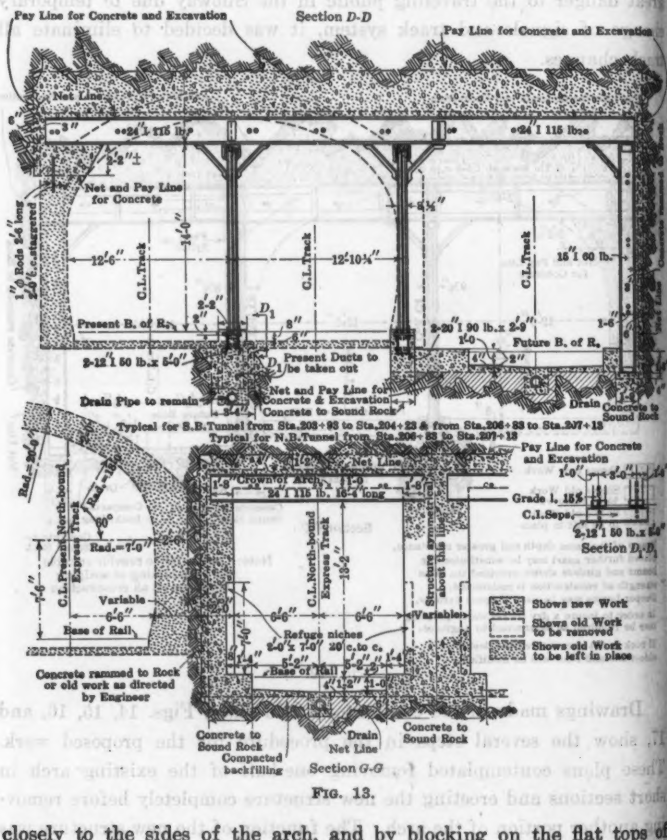
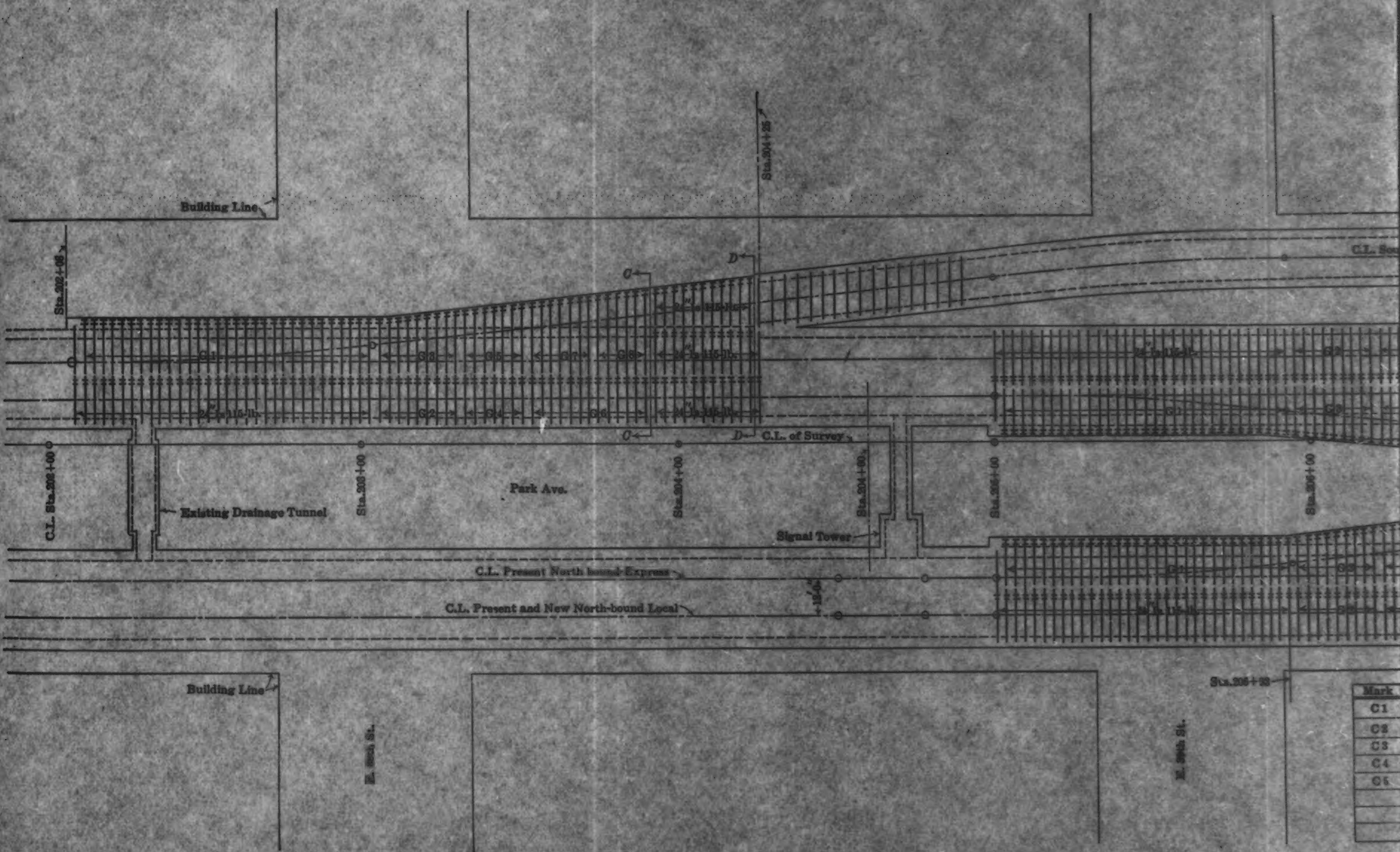


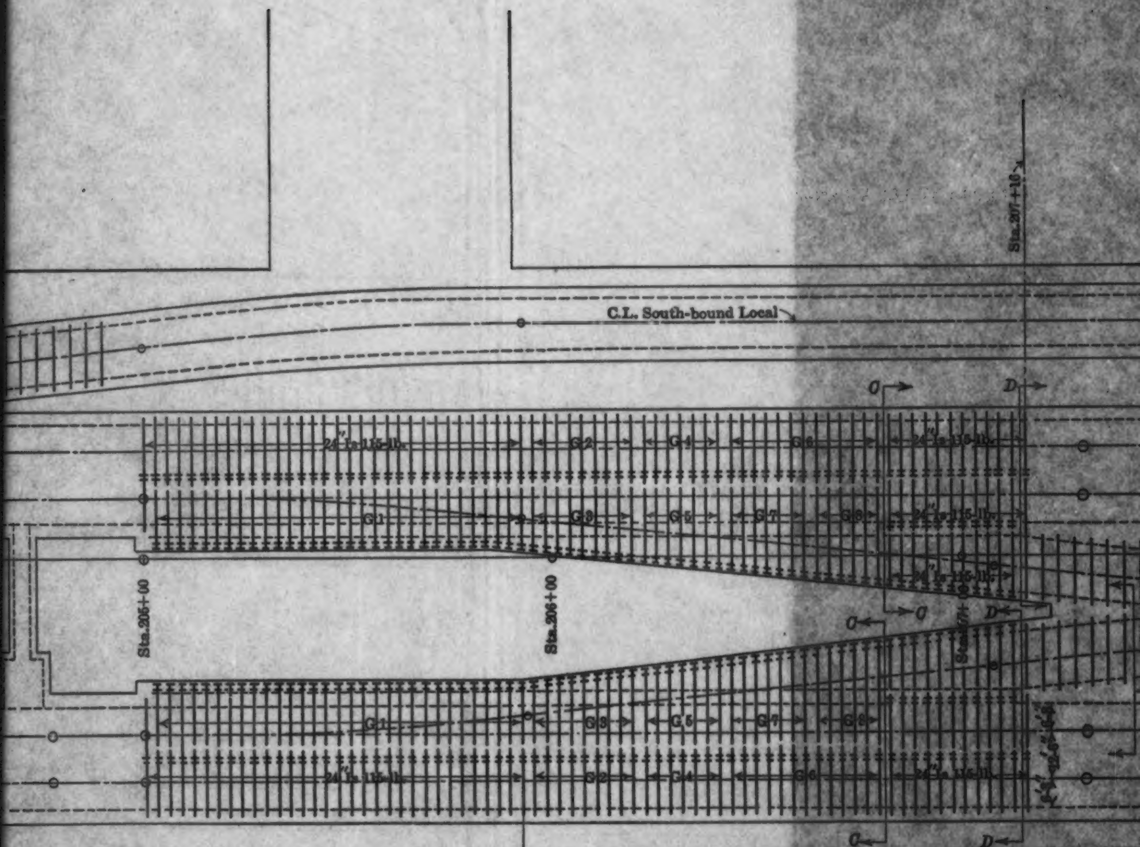
FIG. 13.

closely to the sides of the arch, and by blocking on the flat tops of the shield.

The shield sections were erected while the express trains were not running, between 1.00 and 5.30 A. M. A foreman and seven men constituted the gang that erected the shield. Rubber mats were laid



Building Line
Sta.202+08
Sta.204+25
C.L. Sta.202+00
Sta.203+00
Sta.204+00
Sta.204+60
Sta.205+00
Sta.206+00
C.L. of Survey
Park Ave.
Existing Drainage Tunnel
Signal Tower
C.L. Present North-bound Express
C.L. Present and New North-bound Local
Building Line
Sta.206+33
Mark
C1
C2
C3
C4
C5



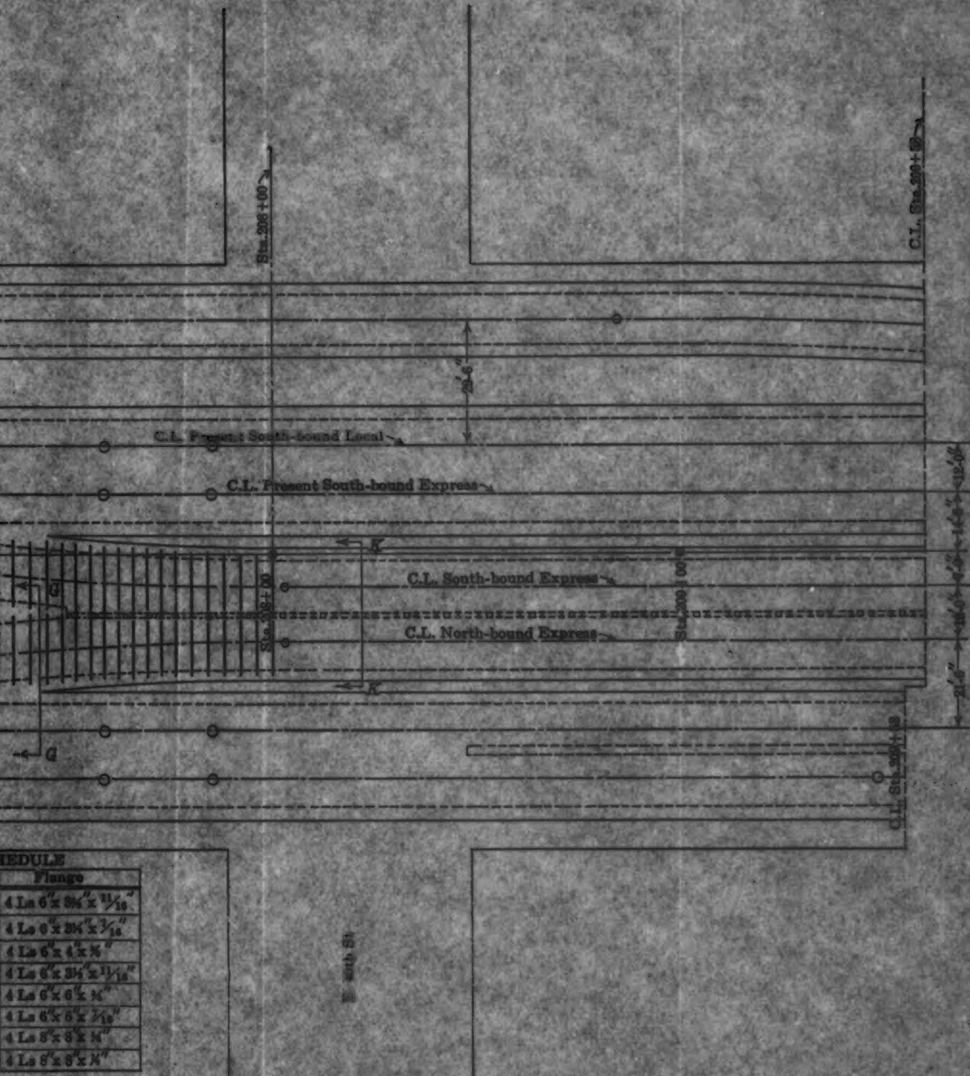
COLUMN SCHEDULE

Mark	Web	Flange
C1	1-8" x 7/16"	4 Ls 6" x 3½" x 7/16"
C2	1-8" x ⅜"	4 Ls 6" x 3½" x ⅜"
C3	1-8" x ⅜"	4 Ls 6" x 3½" x ⅜"
C4	1-8" x 13/16"	4 Ls 6" x 3½" x 13/16"
C5	1-8" x 11/16"	4 Ls 4" x 3" x 11/16"

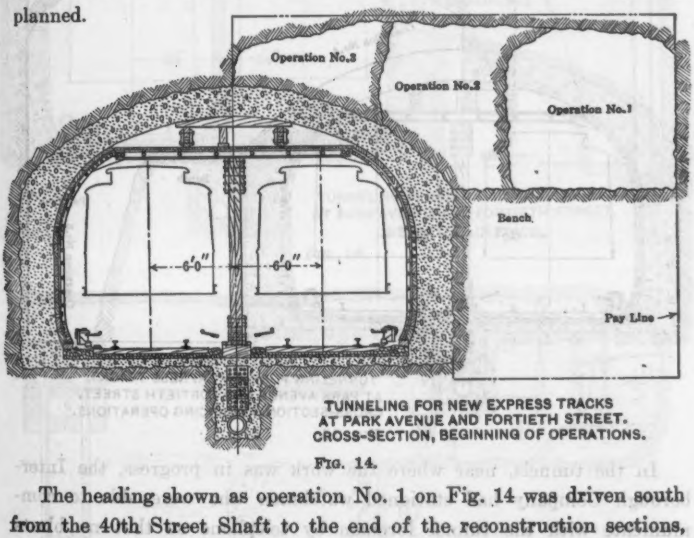
GIRDER SCHED

Mark	Web	F
G1	1-24" x ½"	4 Ls 6
G2	1-30" x ⅜"	4 Ls 6
G3	1-30" x ⅜"	4 Ls 6
G4	1-30" x 11/16"	4 Ls 6
G5	1-30" x ½"	4 Ls 6
G6	1-30" x ¾"	4 Ls 6
G7	1-30" x 9/16"	4 Ls 8
G8	1-30" x ⅜"	4 Ls 8

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1408.
PERRINE ON
SUBWAY CONSTRUCTION PROBLEMS.



on the third-rail guards when the steel ribs were being erected. In order to insure the best possible organization, the men were instructed in their duties, such as placing horses for scaffolds, erection of the ribs, etc., as everything had to be removed and replaced for every train to pass. Consequently, it became necessary to put up a complete steel rib and fasten the parts together between trains running on the local tracks under a 7½-min. headway. The holes in the concrete arch for receiving the expansion bolts, which held the ribs, were drilled by another gang some time before the shields were placed. The different steps in the process of excavation were carried out as originally planned.



TUNNELING FOR NEW EXPRESS TRACKS
AT PARK AVENUE AND FORTIETH STREET.
CROSS-SECTION, BEGINNING OF OPERATIONS.

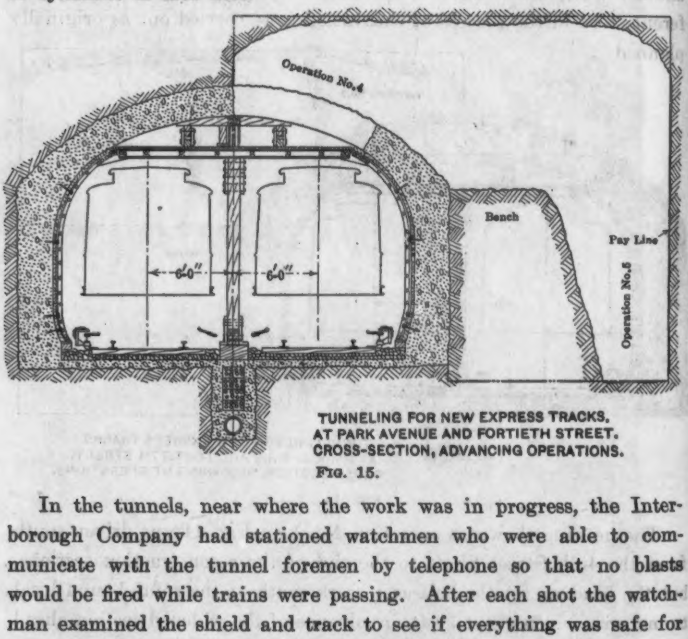
FIG. 14.

The heading shown as operation No. 1 on Fig. 14 was driven south from the 40th Street Shaft to the end of the reconstruction sections, but the heading for the half-arch work south of the south-bound local tunnel was driven about 50 ft. in advance of the widened section ahead of the steel erection. The drills used were Ingersoll-Rand jackhammers (BCR-430). In the heading 8-ft. holes were drilled and 6-ft. holes were driven for operations Nos. 2 and 3, perpendicular to the main heading. The bench was taken out in two lifts, as shown by Fig. 14. Where the concrete arch was thick enough to permit the use of explosives in light charges, this method was used. Otherwise, and for the greater portion of this work, the concrete was removed by plugs and feathers. The plugs were pieces of old drill steel formed into

wedges and driven by Ingersoll-Rand drills (BCR-53) from which the rotating device had been removed. The holes for plugging were drilled about 6 or 8 in. apart, and from 1 to 2 ft. from the face of the concrete to be removed.

The dynamite used was "Red Cross", low-freezing, 40% gelatine, 1¼ in. in diameter, one stick weighing about ½ lb.

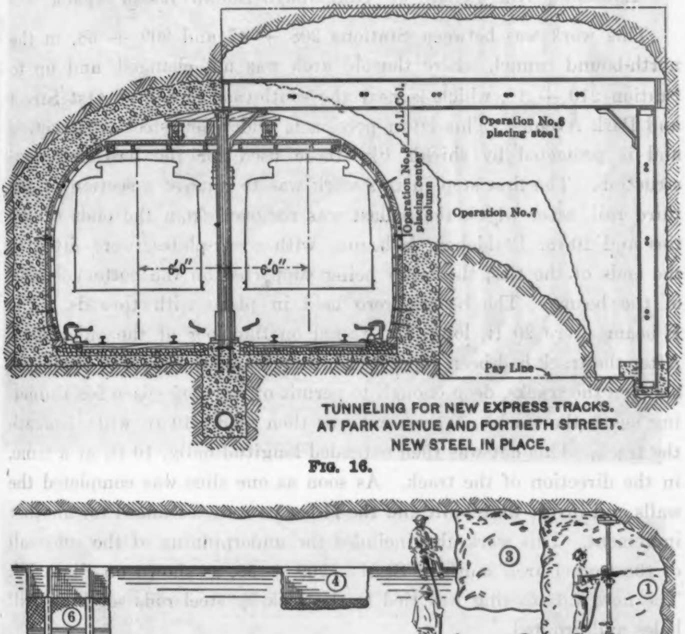
The holes for operations Nos. 2 and 3 were about 12 in. from center to center, two being fired at one time with about one-quarter of a stick of dynamite in each hole.



TUNNELING FOR NEW EXPRESS TRACKS, AT PARK AVENUE AND FORTIETH STREET. CROSS-SECTION, ADVANCING OPERATIONS.
FIG. 15.

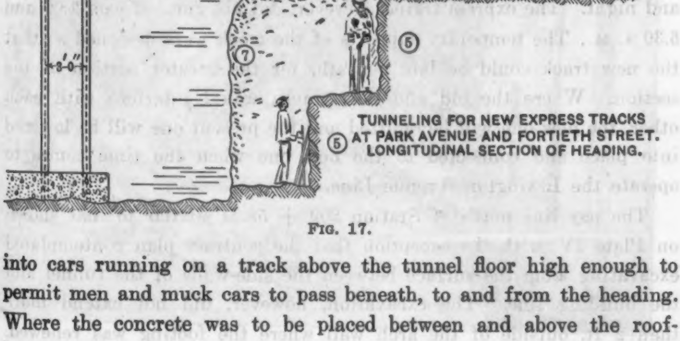
In the tunnels, near where the work was in progress, the Interborough Company had stationed watchmen who were able to communicate with the tunnel foremen by telephone so that no blasts would be fired while trains were passing. After each shot the watchman examined the shield and track to see if everything was safe for the passage of trains. The roof-beams of the new construction are 3 ft. apart. Two beams were placed and concreted each week, and the necessary rock was excavated. The beams, which averaged 2 tons each, were lifted into place with a hoisting engine in the finished tunnel, rigged with a snatch block on the floor ahead and a fall fixed above the roof-beams in the rock by using a head-frame. The concrete forms for the roof were fastened to the roof-beams with hook-bolts. A timber bulkhead was built on the end beam and braced to the

rock face ahead. The concrete was mixed in a ½-cu. yd. mixer in the 40th Street Shaft above the tunnel roof, the materials being fed into the mixer from bins in the shaft above. The mixer discharged



TUNNELING FOR NEW EXPRESS TRACKS.
AT PARK AVENUE AND FORTIETH STREET.
NEW STEEL IN PLACE.

FIG. 16.



TUNNELING FOR NEW EXPRESS TRACKS
AT PARK AVENUE AND FORTIETH STREET.
LONGITUDINAL SECTION OF HEADING.

FIG. 17.

into cars running on a track above the tunnel floor high enough to permit men and muck cars to pass beneath, to and from the heading. Where the concrete was to be placed between and above the roof-beams a pair of timber guides was set up and a light special bucket

for raising the concrete was run up the guides and dumped behind the beams.

LOWERING THE INVERT OF THE NORTH-BOUND LOCAL TRACK.

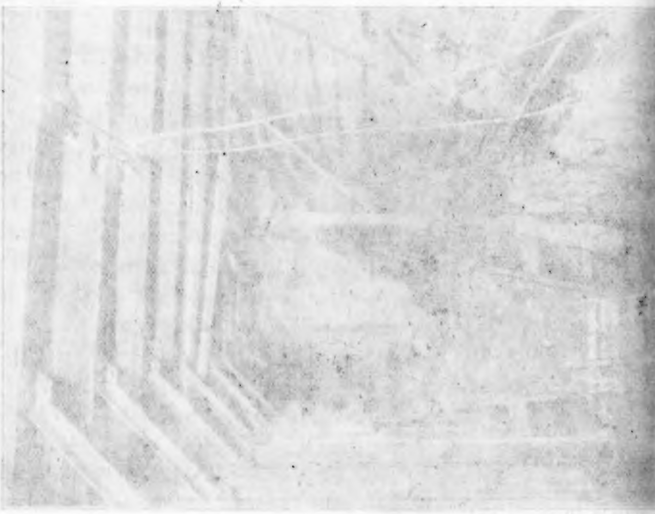
This work was between Stations 208 + 47 and 209 + 58, in the north-bound tunnel, where the old arch was not changed, and up to Station 210 + 18, which is near the south-east corner of 41st Street and Park Avenue. This latter portion is under new steel construction and is protected by shields like those used for the half-arch construction. The first step in this work was to remove a section of the third rail, after which the ballast was removed from the ends of the ties and 10-in. Bethlehem H-beams, with cover-plates, were fitted to the ends of the ties, the latter being supported on the bottom flanges of the beams. The beams were held in place with tie-rods. The H-beams were 20 ft. long, and rested on the floor of the old tunnel. After the track had been supported in this manner, a pit was excavated between the tracks, deep enough to permit of working space for tunneling below the track. A cross-cut was then made, 10 ft. wide, beneath the track. This cut was then extended longitudinally, 10 ft. at a time, in the direction of the track. As soon as one slice was completed the walls and invert were built and the H-beams were advanced for another increment. This work also included the underpinning of the side-wall of the tunnel arch south of Station 209 + 58, as shown on Plate IV. The new wall footing was tied to the rock by steel rods set into drill holes and grouted.

This track, being for local service, was always operated both day and night. The express trains, however, did not run between 1.00 and 5.30 A. M.. The temporary supports of the track were designed so that the new track could be laid beneath, for the greater portion of the section. Where the old and new tracks would interfere with each other, the new one will be omitted and the present one will be lowered into place and connected to the new one when the time comes to operate the Lexington Avenue Line.

The pay line north of Station 209 + 58 is similar to that shown on Plate IV, with the exception that the contract plan contemplated excavating from the surface between the side-walls of the tunnel and the building line. The excavation, however, did not extend more than 2 ft. outside of the arch wall where the footing was renewed.

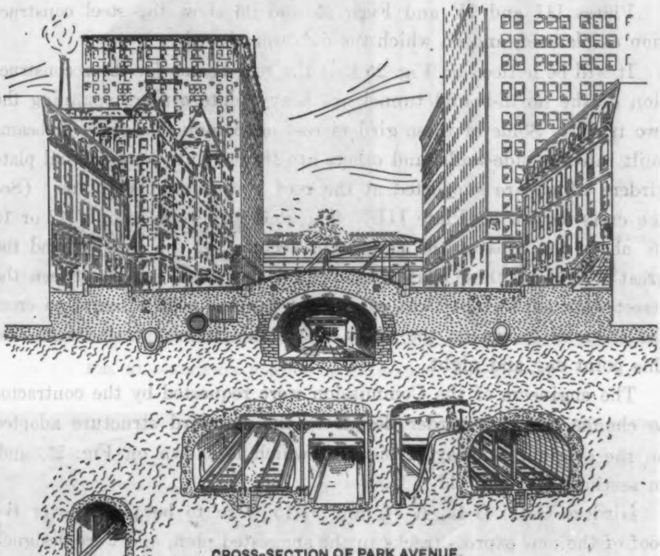
FIG. 18.—SHIELD WITH OLD ARCH REMOVED.

FIG. 19.—REMOVING BENCH.



Before the excavation for the north-bound local track was started, in the Grand Union Hotel lot, two piers of the building at the corner of 41st Street were underpinned to the sub-grade of the cut. Fig. 23 shows the concrete pier at the corner of the building.

The headings for the north-bound and south-bound express track construction north of 40th Street required no timbering. The old



CROSS-SECTION OF PARK AVENUE,
SOUTH OF EAST 42D STREET,
LOOKING NORTH.
The first bent of the new steel construction is shown in place over the old and new express tracks, and excavation is in progress over the north-bound tracks.
FIG. 20.

tunnels were heavily timbered south of 40th Street in places, and this timbering was all removed where it came within the reconstruction lines.

It was found that the grout of the old tunnel had completely filled the spaces between the concrete and the rock. The timbers of the old construction were completely surrounded by the concrete. Extraordinary precautions must have been taken in timbering the tunnel, as in many places where the reconstruction exposed the old

timber the rock was quite sound. Several places, however, were encountered where special care had to be exercised on account of disintegrated rock above the timbers.

CONSTRUCTION OF THE SOUTH-BOUND LOCAL AND EXPRESS TRACKS UNDER THE PRESENT SUBWAY.

Plates III and IV and Figs. 25 and 26 show the steel construction as planned, and on which the bids were based.

It will be noticed on Fig. 25 that the roof beams for the reconstruction of the north-bound tunnel are heavy plate girders spanning the two tracks. Some of these girders rest on columns made of I-beams built into the side-walls, and others are framed into longitudinal plate girders which are supported at the roof elevation by columns. (See the cross-section on Plate III.) The roof girders were only 12 or 15 in. above the tops of the cars. On account of the number and the great weight of these plate girders and the small space between the street deck supports, it was considered dangerous to attempt to erect the steel, as shown on the plans, above the Subway trains that pass this point day and night.

The engineers of the Commission were requested by the contractor to change the plate-girder design to the standard structure adopted on the original Subway, a piece of which is shown on Fig. 25, and, in section, on Plate III.

Girders G-12, G-13, G-14, and G-15 were to be placed over the roof of the new express tracks in the suggested plan, and were designed to support the side-wall columns. Two girders were designed to replace the four shown on Fig. 26 and marked TG-18 and TG-19.

An additional advantage was gained by placing the large girders at the roof level of the new express tunnel, for the reason that they were used to support the steel for the reconstruction of the present north-bound tunnel, thereby permitting the completion of the upper tunnel while the express tunnel was being built.

It was proposed to omit Girders TG-13 over the new south-bound local track and place the roof girders radially with the center line of the track. Drifts were driven under the present Subway for the express and south-bound local. The drifts were 9 ft. high and from 9 to 12 ft. wide, and were timbered where they were directly below the track floors of the Subway. There were from 6 to 9 ft. of rock

FIG. 21.—FINISHED PORTION OF HALF-ARCH CONSTRUCTIÓN.

FIG. 22.—FLOOR OF DIAGONAL STATION, LARGE GIRDERS FOR EXPRESS TUNNEL LYING ON THE FLOOR.

Under the river was quite sound. [illegible]

[illegible]

The engineers of the Commission were requested by the contractor [illegible]

[illegible]

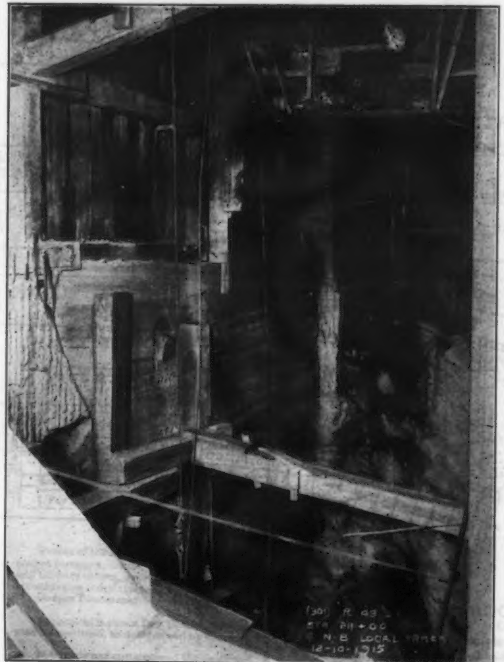


FIG. 23.—UNDERPINNING OF CORNER PIER AT 41ST STREET, AND CUT FOR NORTH-BOUND LOCAL.

FIG. 24.—SECTION OF DRIFT FOR EXPRESS TRACKS, LOOKING NORTH FROM 40TH STREET SHAFT IN "TWIN TUNNEL".

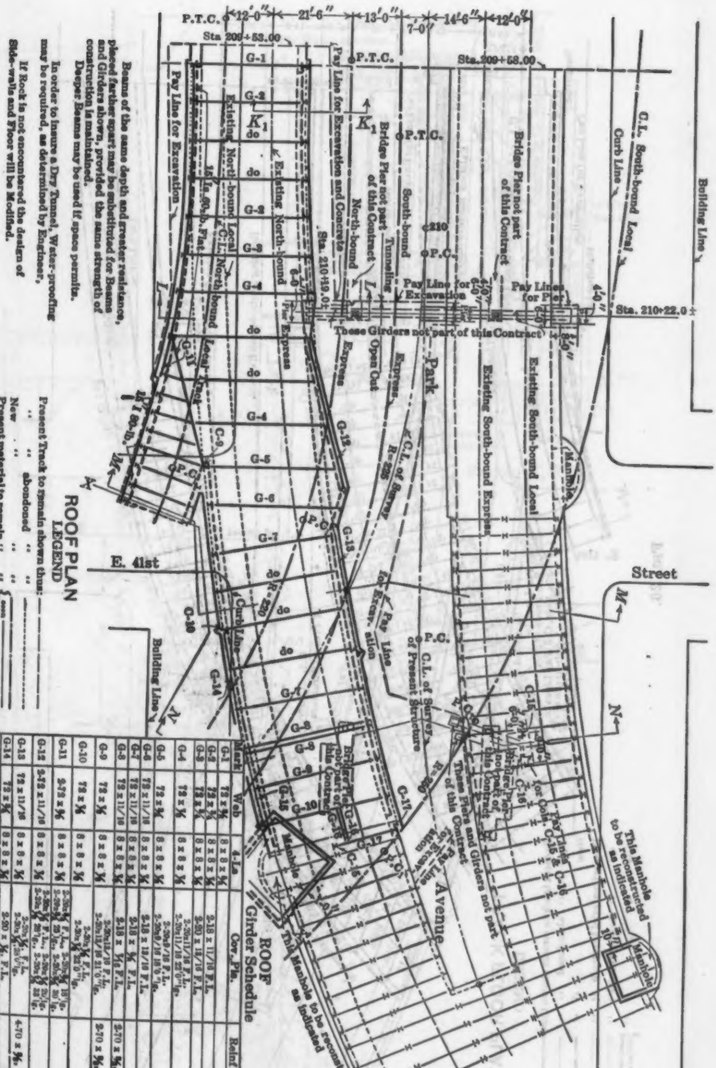

ROOF PLAN
LEGEND
Present Track to remain shown thus:
New
Present material to remain
be removed
ROOF
Girder Schedule
Mark
Web
4-Ls
Cov. Pls.
Reinf. Pls.
Beams of the same depth and greater resistance, placed farther apart may be substituted for Beams and Girders shown, provided the same strength of construction is maintained.
Deeper Beams may be used if space permits.
In order to insure a Dry Tunnel, Water-proofing may be required, as determined by Engineer.
If Rock is not encountered the design of Side-walls and Floor will be Modified.
Pay Line for Excavation
Existing North-bound Local
Existing North-bound Express
Existing South-bound Express
Existing South-bound Local
C.L. South-bound Local
Curb Line
Building Line
Park
Avenue
E. 41st
Street
Open Cut
North-bound Tunneling
South-bound
These Girders not part of this Contract
These Piers and Girders not part of this Contract
This Manhole to be reconstructed as indicated
Sta. 209+53.00
Sta. 209+58.00
Sta. 210+22.0
Sta. 210+19.0
Bridge Pier not part of this Contract
C.L. of Survey of Present Structure
Pay Line for Excavation and Concrete

Fig. 25.

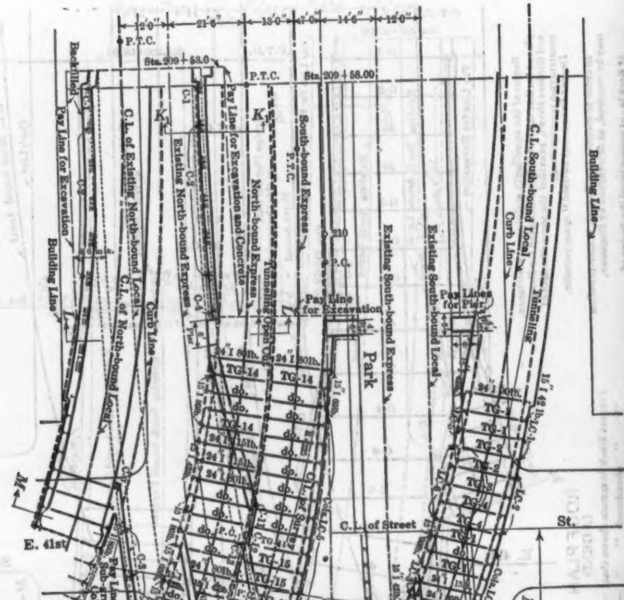
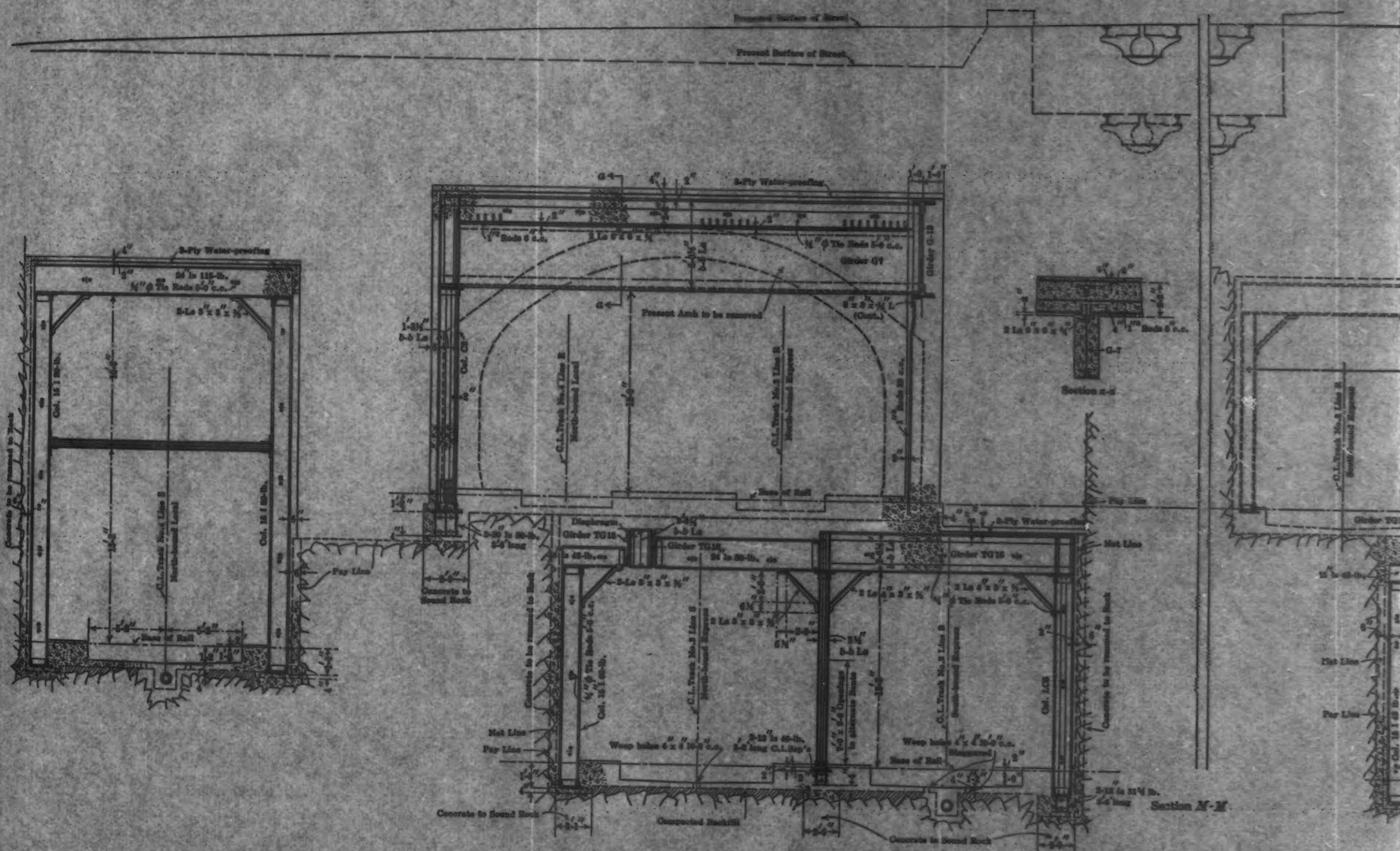
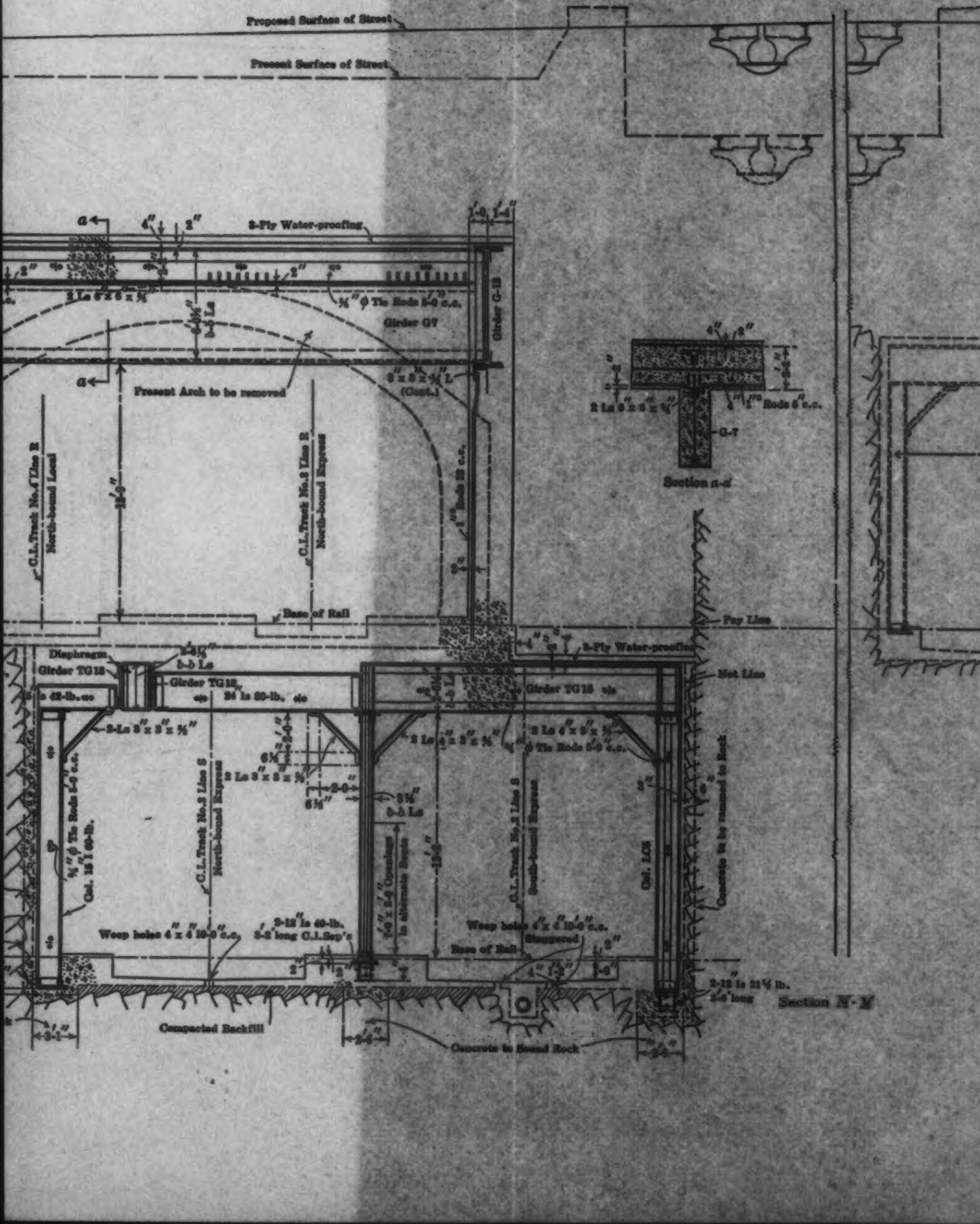

TRACK FLOOR PLAN
LEGEND
Present track to remain shown thus:
" " abandoned " "
New " " "
Present material to remain " "
" " to be removed " "
New " " "
Sta. 209 + 58.00
P.T.C.
C.L. of Existing North-bound Local
C.L. of North-bound Local
Existing North-bound Express
North-bound Express
South-bound Express
Existing South-bound Express
Existing South-bound Local
C.L. South-bound Local
Pay Line for Excavation
Pay Line for Excavation and Concrete
Pay Lines for Pier
Building Line
Curb Line
Backfilled
Tunneling
Tunneling Open Cut
C.L. of Street
Park
Avenue
E. 41st
St.
C.L. of Present Structure
Lower Structure

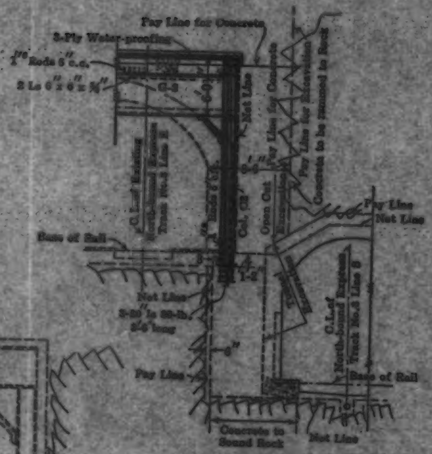
Fig. 26.


Proposed Surface of Street
Present Surface of Street
3-Ply Water-proofing
Present Arch to be removed
Girder G7
Section a-a
Base of Rail
Pay Line
Neat Line
Diaphragm
Concrete to Sound Rock
Compacted Backfill
Section M-M



Proposed Surface of Street
Present Surface of Street
3-Ply Water-proofing
Girder G7
Girder G-13
Present Arch to be removed
Section a-a
C.L.Track No.4 Line 2 North-bound Local
C.L.Track No.3 Line 2 North-bound Express
Base of Rail
Pay Line
Net Line
Diaphragm
Girder TG18
Girder TG16
C.L.Track No.3 Line 3 North-bound Express
C.L.Track No.2 Line 3 South-bound Express
Weep holes 4" x 4" 10-0" c.c.
Weep holes 4" x 4" 10-0" c.c. Staggered
Compacted Backfill
Concrete to Sound Rock
Concrete to be rammed to Rock
Section N-N

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1408.
PERRINE ON
SUBWAY CONSTRUCTION PROBLEMS.



Section K_1-K_1

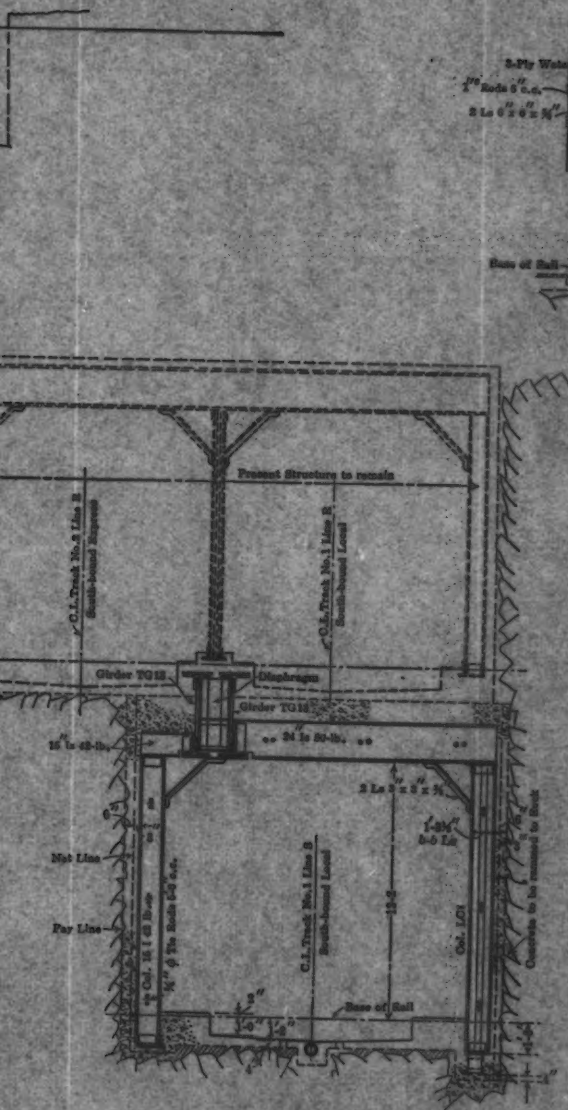


Fig. 27.—Express Drift, Looking South.

Fig. 28.—Express Drift with Large Girder Passing Through.

FIG. 29.—ARCH OF NORTH-BOUND TUNNEL DRILLED AND REMOVED WITH PLUGS AND FEATHERS.

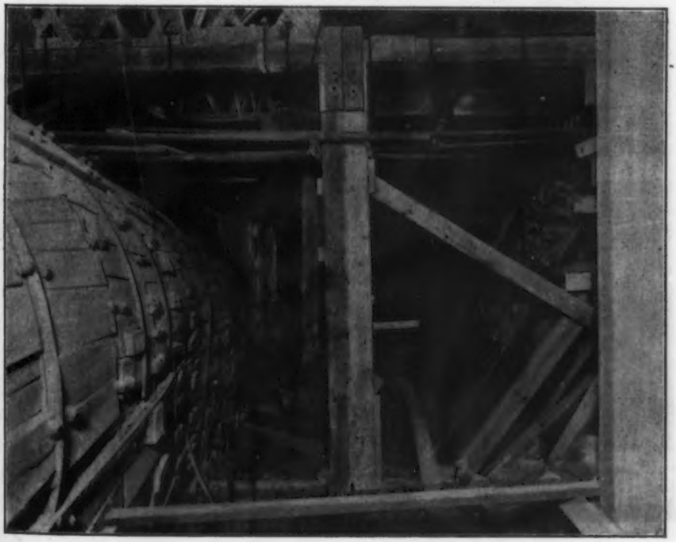


FIG. 30.—EXPRESS CUT. DRILLING FOR TRENCH TO RECEIVE LARGE GIRDERS.

FIG. 31.—TRENCH FOR GIRDER BETWEEN THE TWO NORTH-BOUND TRACKS.

FIG. 32.—NORTHEASTERLY GIRDER IN PLACE, SHOWING CUT IN ROCK TO RECEIVE COLUMNS.

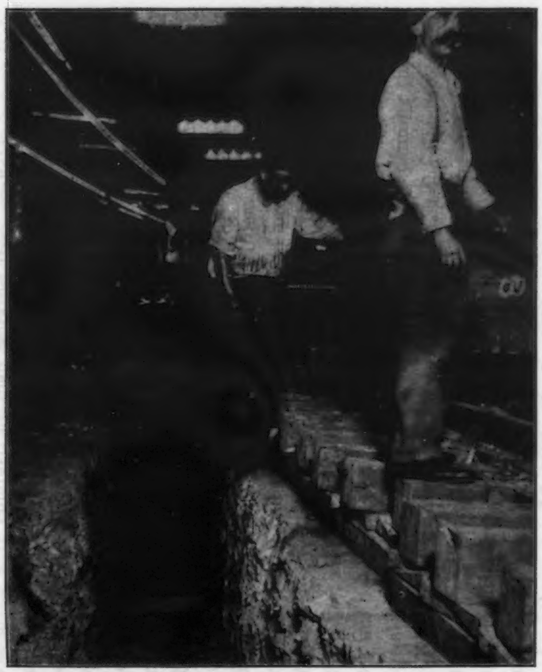


FIG. 31.—TRENCH FOR GIRDER BETWEEN THE TWO NORTH-BOUND TRACKS.

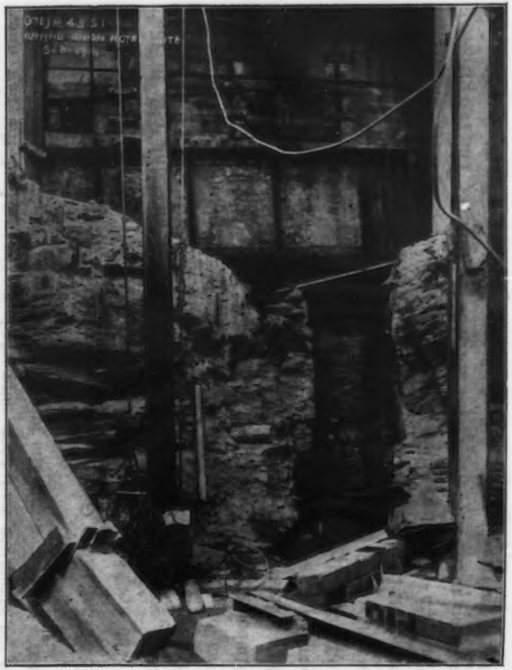


FIG. 32.—NORTHEASTERLY GIRDER IN PLACE, SHOWING CUT IN ROCK TO RECEIVE COLUMNS.

left between the roof of the drift and the bottom of the track floor. The drifts were enlarged by men working from scaffolds, using jack-hammer drills. A band of rock was excavated to receive one or two bents of steel, and these were at once erected and concreted. This operation was carried on in four places by two gangs, each alternating between two points in order to allow the concrete to set while drilling for a new band. The contract plans contemplated the removal of the concrete floor of the old tunnels, in order to place the new steel, but, by the adopted plan, it was possible to place the new steel without disturbing the invert (excepting where deep girders occurred) or interfering with the Subway traffic. As an extra precaution, the tracks of the Subway were supported by Bethlehem H-beams at the ends of the ties, and these were placed so as to span the space where the rock was being removed below. The columns of the Subway structure were also supported temporarily by the underpinning spanning the cut.

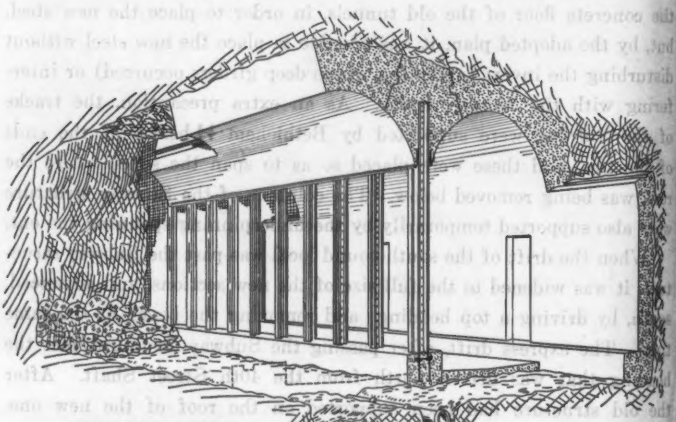
When the drift of the south-bound local was past the Subway structure, it was widened to the full size of the new sections, and advanced, south, by driving a top heading and removing the bench at the same time. The express drift, after passing the Subway structure, met the heading that was driven north from the 40th Street Shaft. After the old structure had been supported on the roof of the new one, grout was pumped into the joint between them.

Erecting Large Girders and Columns for the Express Tunnels.

The first operation in connection with the erection of the steel was to excavate the trenches for the girders, and sink pits in the rock, from the level of the Subway to the sub-grade of the grillages. Seven of these columns were just outside of the walls of the old north-bound tunnel and three were between the Subway tracks. The large girders that were at the east of the old Subway at the points where Girders G-14 and G-15 are shown on Fig. 25, were raised into position from the open cut with a gin-pole and tackle and rolled into place. All the other large girders were run through the express track drift from the station floor, and south in the express tunnel far enough to permit them to be pulled endwise up into the open excavation at the west side of the north-bound tunnel. The two girders between the tracks were passed, from the express cut, through an

opening in the shield, across the express track, and placed at night when the express service was not in operation.

The trench between the tracks and the pits, for the columns that were to support the large girders, was ready some time before the girders arrived. Several of the pits were stoped up from the express drifts, and others were sunk from the bottom of the open cut and the tunnel floor.



ORDER OF CONSTRUCTION.

1.-Drift driven on center line 8 ft. wide and full height.
2.-Grillages placed and concreted.
3.-Columns set and "umbrella section" of concrete built up to rock to act as support while widening excavation.
4.-Drift widened on one side of columns, and wall built
5.-Arch built between side-wall and "umbrella section"
6 and 7.-Same as 5 and 6, but for opposite side.
8.-Invert built.
9.-Curtain-wall between columns placed.

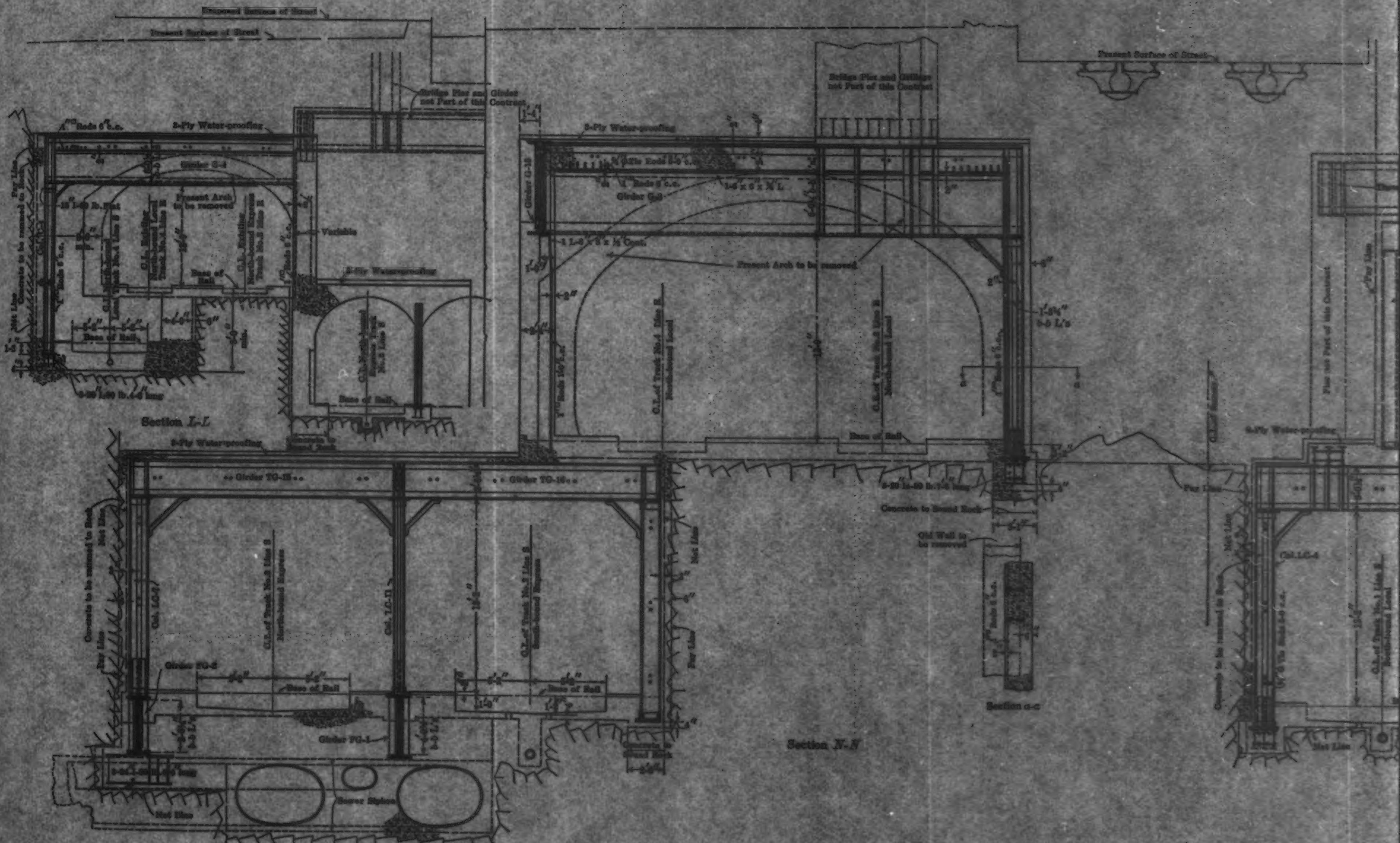
DIFFERENT STEPS IN THE CONSTRUCTION OF THE EXPRESS TUNNEL UNDER PARK AVENUE FROM 40TH TO 41ST STREET

Width between side walls, 26 ft. 0 in.
Height from floor to arch, 15 ft. 5 in.
Center columns 5 ft. 0 in. apart.

FIG. 33.

The weights of the girders west of the tracks are 70 350 and 28 900 lb.; those between the tracks, 36 500 and 20 500 lb.; and the two east of the tracks, 16 600 and 15 200 lb.

Personnel.—The personnel was as follows: Representing the Public Service Commission of the First District of New York: Alfred Craven, M. Am. Soc. C. E., Chief Engineer; Daniel L. Turner, M. Am. Soc. C. E., Acting Chief Engineer; Robert Ridgway, M. Am. Soc. C. E., Engineer of Subway Construction; John H. Myers, Assoc. M. Am. Soc. C. E., Engineer, Second Division; Robert H. Jacobs, Assoc.


Proposed Surface of Street
Present Surface of Street
Bridge Pier and Girder not Part of this Contract
3-Ply Water-proofing
Girder G-4
Present Arch to be removed
Variable
Base of Rail
Section L-L
Girder G-8
Concrete to Sound Rock
Girder TG-15
Girder TG-16
Girder TG-2
Girder TG-1
Sewer Siphon
Neat Line
Pay Line
Old Wall to be removed
Section a-a
Section N-N

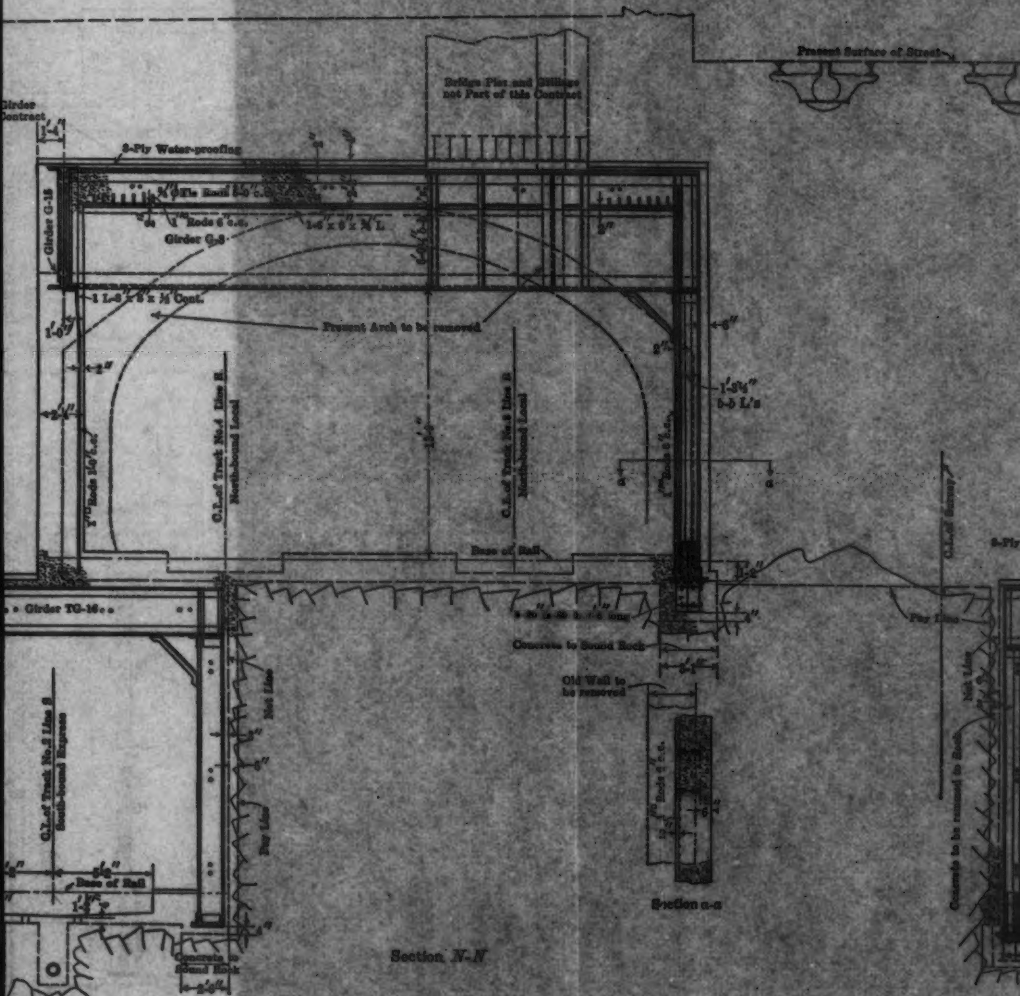
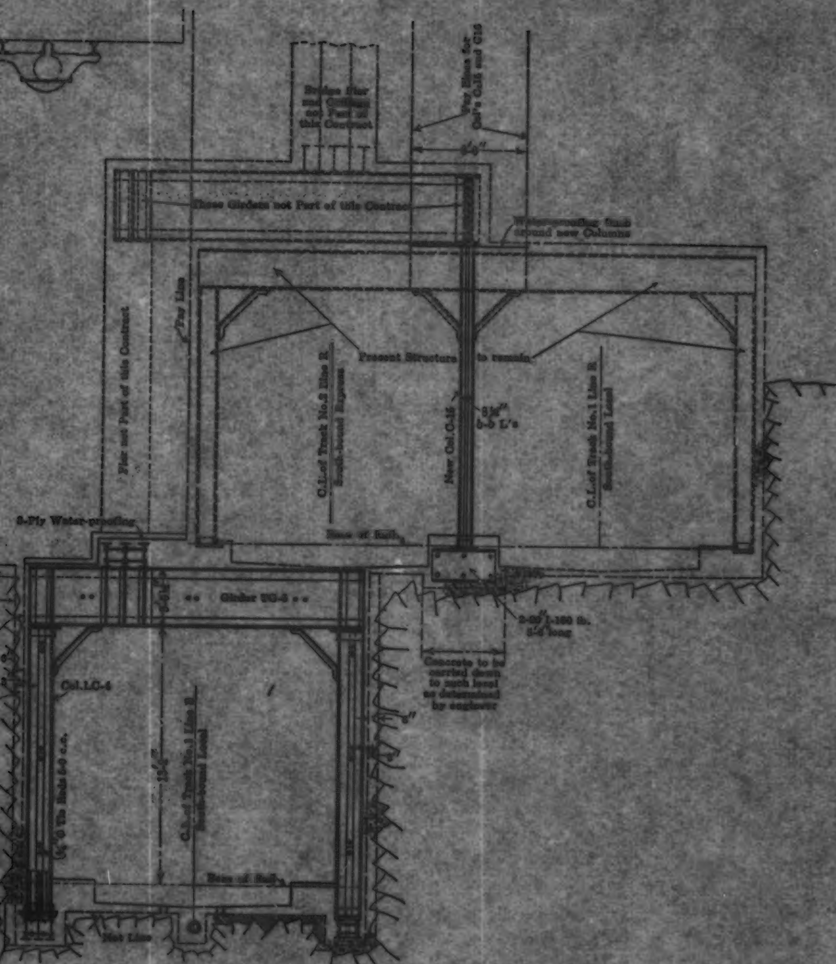

Present Surface of Street
Bridge Piers and Grillage not Part of this Contract
3-Ply Water-proofing
Girder G-18
Girder G-8
Present Arch to be removed
C.L. of Track No.4 Line 2 North-bound Local
Base of Rail
Girder TG-16
Concrete to Sound Rock
Old Wall to be removed
Section a-a
Neat Line
Pay Line
C.L. of Subway
Concrete to be rammed to Rock
Section N-N

PLATE IV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1408.
PERRINE ON
SUBWAY CONSTRUCTION PROBLEMS.



M.
Sch
nee
Cor
Eng
Geo
Mr.

M. Am. Soc. C. E., Senior Assistant Division Engineer; Mr. Stephen Schmidt, Assistant Engineer; and Mr. Melvin Miller, Assistant Engineer. Representing the Rapid Transit Subway Construction Company, Contractor: George H. Pegram, President, Am. Soc. C. E., Chief Engineer; Robert A. Shailer, M. Am. Soc. C. E., Tunnel Engineer; George Perrine, M. Am. Soc. C. E., Assistant Tunnel Engineer; and Mr. Thomas McCormick, General Superintendent.

DISCUSSION

Mr. Myers.

JOHN HAYS MYERS,* ASSOC. M. AM. SOC. C. E.—It is well that some one has placed on record a description of the work done in connecting the new Lexington Avenue Subway with the existing subway on Park Avenue, as well as of that incidental thereto. Much of it was done either under, over, or alongside what is without exception the densest railroad traffic in the world. In fact, the men in charge of operation speak of the tracks as being "saturated with trains." Although the problems met are rare, and peculiar to construction work carried on near moving trains, nevertheless, the principles involved and the methods used in solving these problems are well worthy of record. The author, therefore, deserves the thanks of his fellow-engineers for contributing this paper to the *Transactions* of the Society.

It is to be regretted, however, that he did not go into more detail, because much more could have been written, with profit to the Profession, on this interesting piece of construction and on the methods devised and carried out so successfully. The speaker desires, therefore, to amplify certain matters contained in the paper and to bring out others which the author has not mentioned.

The engineers of the Public Service Commission realized before the contract was advertised that, to use the language of the author, the work involved was not by any means, "the standard construction in the New York subways." Therefore, much study was given to drawing up the contract, writing the specifications, and making the plans. That the time devoted to this study was well spent is shown by the fact that the work has been completed with few and minor departures from the contract plans, and that few questions have arisen which were not covered by the provisions of the contract.

The author's remark that the standard construction in the New York subways will not be dealt with in this paper has apparently precluded any reference to the Diagonal Station proper, other than to the method of excavating, on page 280. The Diagonal Station, so-called because of its location diagonally across the block east of Park Avenue between 41st and 42d Streets, is anything but standard subway construction. Although, as far as subway traffic is concerned, it is an ordinary two-island-platform station, similar to the present Grand Central Subway Station, its foundations, columns, and roof are designed so as to permit of placing above it a tall and heavy building, in order that the property condemned by the City of New York, 197 ft. 6 in. on Park Avenue and 230 ft. on 41st and 42d Streets, may be fully utilized for building purposes. To accomplish this, special attention was given to the foundations and to the design of grillages,

* New York City.

columns, and roof members, members of extra strength being disposed so as to anticipate, as far as possible, the needs of any future building operations. Where a building column, because of its location, could not be carried directly on one of the station columns, heavy twin girders were introduced in the station roof to take the load of such column and carry it to two of the station columns. Mr. Myers.

The paragraph on sewers, on page 280, refers to the sewer construction beneath the subway at 41st Street and eastward under that street. Some sewer work was done, also, on the east side of Park Avenue, between 41st and 42d Streets, which continued northwestward under 42d Street to a point near Vanderbilt Avenue. It was first planned to build the Park Avenue sewer on the west side of the avenue, but, owing to the fact that it would have necessitated tunneling adjacent to the Hotel Belmont, in what was likely to be shattered rock beneath the floor of the existing south-bound Park Avenue Subway, the Commission's engineers changed the plan, and the sewer was built on the east side. A portion of the excavation consisted merely of a trench in rock below the general level of the excavation for the Diagonal Station. North of the excavation for the Diagonal Station, this sewer was built in tunnel and, when put in service, permitted the contractor to discontinue the use of the sewer by-pass which the author mentions in connection with his description of the large shaft connecting with the Queensborough Subway beneath 42d Street.

The references to duct lines, on pages 280 and 282, do not bring out the fact that the existing duct lines between the tracks in the north- and south-bound Park Avenue tunnels were a vital link in the subway operation, inasmuch as they carried cables which furnished power for both operation and lighting. Realizing this, the engineers of the Commission provided in the contract that, in addition to building the new duct lines which the author describes, the contractor should draw into these lines all necessary cables to replace those which were removed from the existing tunnels. This work was accomplished successfully without any interruption of train service. The contract provided that payment for this cable work should be upon a cost-plus-percentage basis.

The author states that the duct line near Fifth Avenue was built on account of the change in the tracks in 42d Street in front of the Grand Central Terminal. It would be clearer, perhaps, to say that this duct line was built because, in changing these tracks, or rather extending them eastward to form a stub-end terminal, the side-wall of the existing subway was removed and with it the duct bank which carried the power cables. The purpose of the duct line at Fifth Avenue was to make a connection between the lines on the north side of the 42d Street Subway and those on the south side, and thereby permit of carrying cables past the gap in the duct line, resulting from the

Mr. Myers. removal of this side-wall. The new duct line was built by tunneling beneath 42d Street east of Fifth Avenue.

On pages 282 to 288 the author describes the very successful work of lowering the grade of the Steinway Tunnel tubes and converting them into a wide tunnel containing two tracks with an island platform. It should be recorded here that this was similar to a piece of work carried on previously by the same contractors in a part of the Queensborough Subway under 42d Street east of First Avenue. This work is very well described and illustrated in a paper* by Mr. Melville S. Miller, the engineer in charge of that work as well as of the Queensborough Subway work described by the author.

The author describes, on page 290, what he terms the "half arch construction." He first describes and illustrates quite fully the contract plans covering this portion of the work, and then the modifications of these plans and the method of executing them. The modifications proposed by the contractor are no doubt an improvement on those shown by the contract drawings. The design is similar, to those shown in the contract, the improvement consisting principally in the provision that no work was to be done over one of the existing tracks where the half arch was left in place. Furthermore, by raising the roof members to receive the thrust of this half arch, these members were removed farther from the tops of the cars, and the insertion of a shield permitted the operation of the trains over the regular tracks without routing them temporarily over other tracks by cross-overs. It should be remembered, however, that the design shown in the contract plans was based on a similar design made by the Commission's Engineering Corps and carried out successfully by the same contractor at the 168th Street and 191st Street Subway Stations. In these cases, the old tunnel arch was carried on new steel members placed beneath it, and the side-wall supporting the arch was removed to permit of the introduction of new or extended platforms.

In referring to the sequence of operations illustrated in Fig. 14, the author does not bring out the fact that, in Operation No. 1, namely, the driving of a longitudinal drift parallel to the existing tunnels, the elevation of the bottom of this drift was fixed so that the material which took the thrust of the arch would not be removed until, by Operations Nos. 2, 3, and 4, the greater part of this thrust had been removed. In removing the material and the tunnel arch, it was first intended to do it in alternate bands, leaving the intervening material in place. This, however, did not prove very successful, and it was found that, instead of removing the material in alternate bands or cross-drifts, it could be removed in successive bands, that is, the rock overlying the existing tunnel was removed in bands ordinarily not exceeding 7 ft. in width. The new structure, consisting

* *Public Service Record*, May, 1915.

of steel roof beams with concrete between, was then built in this space and the overlying material was caught up on the concrete before another band of material was removed. In this way, the subway trains were never exposed for a distance of more than 11 ft., and ordinarily not more than 7 ft. As a rule, the rock spanning this space between the new structure and the old arch required no support. In the few cases where it was thought that some of it might fall, it was readily kept in place by temporary longitudinal needles with one end resting on the completed structure and the other on the arch which had not yet been removed. Mr. Myers.

In the paragraph headed "Excavation", the speaker believes that the author, in referring to the portion of the work in Park Avenue between 42d and 40th Streets, meant to say "42d and 41st Streets", because the open-cut work in Park Avenue extended, roughly speaking, from a point south of 42d Street to a point somewhat south of 41st Street. This paragraph gives the impression that the south-bound local track was the only one where tunneling methods were used, although the difficult work which the speaker has just mentioned was carried on by such methods under Park Avenue south and north of 40th Street.

The very complete diagram, Fig. 1, shows the location of the shaft used to carry on the express tunnel work on Park Avenue north and south of 40th Street. This shaft is interesting from the fact that its east and west dimensions were limited, its westerly limit near the surface of Park Avenue being determined by the wall of the tunnel through which the Park Avenue surface cars run, and near its bottom its easterly limit was determined by the wall of the existing north-bound tunnel. The excellent cross-section of the tunnels under Park Avenue, Fig. 20, illustrates these limits very well.

On page 298, the author speaks of the erection of the steel in the "half arch section". As the new tracks approached the existing ones, the space in which steel could be handled became more confined and, in some parts of this work, two hoisting engines were used to erect this steel, one being placed in the finished work and the other in the drift marked "Operation No. 1" in Fig. 14.

The author, in describing the "half arch section", goes into considerable detail in a discussion of the contract plans, and then shows how these plans were modified in carrying on the work. It is to be regretted that he did not treat, in a similar way, the construction of the south-bound local and express tracks under the present subway, and the rebuilding of the present north-bound structure, which he describes on pages 304 to 318. He gives the design, as shown on the contract plans, but does not give drawings illustrating the work as actually carried out. The difference in the plans, however, is not great.

Mr. Myers. The possibility of readily building what constitutes a plate-girder bridge to carry the existing tracks over the new tracks passing beneath them was realized in getting up the contract plans. It was also realized that modifications in detail might be asked for by the successful bidder, and inasmuch as it meant the redistribution of practically the same tonnage of steel, it was thought that the plans were sufficiently comprehensive to permit of intelligent bidding.

On page 317, the author states that when the drift of the south-bound local had passed the subway structure, it was widened to the full size of the new sections. Figs. 25 and 26 illustrate this point, and indicate that the south-bound local tunnel passed through a duct manhole or splicing chamber on the west side of the existing structure near 41st Street. The bottom of this manhole was below the general floor level of the existing tunnel, and contained live power cables, and, until these cables could be removed, it was necessary to carry the south-bound local tunnel in a bottom heading. Figs. 25 and 26 do not show the walls of the vault of the Hotel Belmont, which walls followed the west curb of Park Avenue and the center line of 41st Street. The southeast corner of this vault was about at the west line of the south-bound tunnel. As a matter of fact, the brickwork of these walls was uncovered in driving this tunnel, but the work was carried by them without damage to them or to the duct manhole and the cables in it.

The speaker thinks it proper to make mention here of the great value in this work of the records of the original subway work done in this neighborhood. From them, the Commission's engineers were able to predict with considerable accuracy what would be encountered in the way of construction and of material to be excavated.

In reference to Fig. 33, showing the method of erecting the express tunnel under Park Avenue, it should be pointed out that this design was well adapted to sectional construction. It is practically the same as that used in the construction of parts of the express tracks beneath Lexington Avenue, which work was described in a paper* by Israel V. Werbin, Assoc. M. Am. Soc. C. E. From a theoretical viewpoint, the question might be asked: When half of this structure is built, what balances the thrust of the arch? The speaker's theory with regard to this is that unless the rock is "heavy", there is little or no thrust; but, in any event, by building first what the author terms the "umbrella", the concrete engages with the roughness of the rock overhead and thus takes up what thrust there is, although it is possible, in using this construction, to put in temporary struts between the "umbrella" and the unexcavated rock on the other side of the tunnel until such time as the remaining arch is completed.

* "Tunnel Work on Sections 8, 9, 10 and 11, Broadway-Lexington Avenue Subway, New York City," *Transactions*, Am. Soc. C. E., Vol. LXXXI, p. 341.

It would be improper if mention were not made of the use of the small drill commonly known on the work as the "Jap". In carrying the new tracks beneath the existing ones, the author refers to sinking pits or shafts, or else stoping them up from below, in which to erect various columns. In this work and in excavating rock very close to the completed structure, these drills were of very great advantage. It is difficult to imagine that it would have been possible to do this work with the old tripod drill which was in common use about 15 years ago on the subway construction. Mr. Myers.

Mr. Quimby has pointed out that the water-proofing of the Queensborough Station arch beneath 42d Street was omitted, although it was shown on the contract drawings. An effort was made to water-proof this arch according to the contract drawings, but it was finally concluded that it was not feasible.

On pages 286 and 287 the author has described the method of excavating and placing the concrete for the station arch. As there described, this arch was built generally in alternate bands, and had it been possible to obtain a water-proofing compound which would not have emitted fumes to any great extent, it might have been possible to water-proof, say, the 1st, 3d, 5th, etc., bands. It might also have been possible to water-proof the haunches of the 2d, 4th, 6th, etc., bands; but the speaker has never known of a practical solution of the problem of how to water-proof the closures. As he sees the problem, it would be necessary to place the water-proofing and then build the arch beneath it. Considerable thought having been given to water-proofing this arch, it was decided, as the author states, to omit it. However, the concrete of the arch was placed as close as possible to the rock. Numerous grout pipes were placed, and whatever space remained between the concrete and the rock was filled with grout. Later, the surface of the concrete was completely removed from the soffit of the arch by using pneumatic hammers, and a dense and well-troweled coat of plaster, containing some water-proofing compound, was applied. Since this work was done, a part of the arch has been removed in order to permit the construction of a passageway or ramp connecting the Queensborough and Diagonal Stations. This has permitted some water to flow into the Queensborough Station, but this condition, of course, has no connection with the water-tightness of the Steinway arch which, considering all the conditions, is fairly tight.

Clarence E. Carpenter,* Assoc. M. Am. Soc. C. E.—The members of this Society should be congratulated on having presented to them a clear and concise description of the work of constructing the portion of the Lexington Avenue Subway south of the north side of 42d Street, including the connection of that subway with the present one in Park Mr. Carpenter.

* Yonkers, N. Y.

Mr. Carpenter.

Avenue, and also the extension of the Queensborough Subway from a point about 100 ft. east of Lexington Avenue to Vanderbilt Avenue. Many of the members are doubtless familiar in a general way with the scope of this work, but its difficulties and hazardous character can only be realized by examining the plans and considering the conditions under which it was done. It may be well to call attention to the fact that 1908 trains pass through the subway every week day, over the portion which was reconstructed in Park Avenue, carrying about 1 000 000 passengers. The interval between trains on each of the four tracks during the rush hours is 1 min. 48 sec., and the longest interval between trains on the local tracks, which occurs for a few hours after midnight, is 7½ min.

The speaker's position, as Engineer of Maintenance of Way with the Interborough Rapid Transit Company, has made it necessary for him to follow the progress of many of the large sub-surface construction jobs under way in New York City during the past 12 years, much of this work having been carried on in close proximity to either the elevated or subway lines. He can say, without hesitation, that not one of the jobs he has seen has seemed to him to be so difficult to perform at a reasonable cost and with safety to the traveling public as the one under consideration. The speaker does not know what the contractor's expenditures are, but he can state that the work has been done safely and well. This satisfactory result was only obtained by the closest attention to details, the exercise of good judgment, and the full co-operation of the operating men of the Interborough Rapid Transit Company.

The contractor's engineers deserve credit for suggesting the "Half Arch Construction", which, as compared with the original design for that portion, made the work much simpler. In the case of the express track connections, the removal of only the half of the arch over the old express tracks was required, and on these tracks trains are not run between about 1.00 and 5.30 A. M.

The shield erected on the interior of the old tunnel was an ingenious design, and worked out very well. The author states that it was not designed to support large masses of displaced concrete. As a matter of fact, it was made about as strong as the space would permit. The clearance between the shield and the edge of a car roof was so small that, as an added precaution, a steel plate was mounted on the inside of the shield, longitudinally with the tunnel, in order to present a continuous smooth surface on which the corner of the car roof would slide if a car spring should break. The manner of setting and anchoring the bottom of the steel angle ribs was also given close attention, for, if displaced, they might have come in contact with the third rail. Such an occurrence would cause a heavy ground, and possibly set fire to the wooden shield.

The change of the plate-girder design to the standard construction of the old subway, where the new subway tracks cross under the old north-bound tunnel, facilitated the work very much. The heavy plate girders designed for the roof of the old tunnel were placed in the floor of that tunnel, where they could be handled, and where they are used to support the I-beams, which were placed one by one under the track as the concrete invert was removed. The shield built around the interior of the old north-bound tunnel at this location differed somewhat from that erected for the "half arch construction". Center supporting posts between the tracks could not be used, as they would interfere with the placing of the heavy girders previously referred to; consequently, the horizontal top portion of the shield was suspended by bolts from the girders supporting the street decking. The top of the shield consisted of 6 by 10-in. wooden beams, placed in pairs, and about 6 ft. from center to center, on which were laid two courses of 2-in. plank. Each pair of beams was supported by three suspender bolts, $1\frac{1}{4}$ in. in diameter, one at each end of the beam, and one in the center. Mr. Carpenter.

The author has referred to the reconstruction of the floor of the old subway under the north-bound local track and the temporary support of that track designed to permit of lowering the track quickly to conform to the grade required to enter the new Grand Central Station of the Lexington Avenue line at the same elevation as the three other tracks at that station. The speaker does not contemplate lowering the track, but plans to divert the north-bound local trains to the north-bound express track between 33d Street and the new Grand Central Station, during the period when express trains are not run. The local track will then be taken up and relaid to the new grade, using new rails which have previously been curved to suit the new alignment and having the new standard section.

One of the main reasons for the success of this work was the method of excavating only as much as would permit of the erection of an individual steel member, which was promptly placed in position and used to support the load it was designed to carry. This method insured the safety of the subway, and made it unnecessary to use a large quantity of timber falsework.

Robert A. Shailer,* M. Am. Soc. C. E.—It is always a great satisfaction to the engineer to have the difficulties of the construction work under his charge appreciated. The several members of the Society who have taken part in the discussion have the speaker's sincere thanks, and to Mr. Myers, especially, all are indebted for his elaboration of the paper. Mr. Shailer.

The contractors have had the most hearty co-operation from the Public Service Engineers, and they deserve great credit for giving

* New York City.

Mr. Shailer. lines and grades under very trying conditions, due to subway trains passing constantly alongside and often directly over them. The surveys were made so accurately that there was no difficulty in joining the new steelwork to the steel previously erected by the New York Central Railroad Company, on the site of the Hotel Commodore, nor with the steelwork of the Queensborough Elevator Shaft.

Mr. Ridgway. ROBERT RIDGWAY,* M. AM. SOC. C. E.—It is interesting to note that the work described in this paper was one of the most complicated and difficult of the eighty or more sections of construction included in the new Dual System of Rapid Transit for New York City. The estimated value of the work described by the author is about $3 100 000, the cost of the whole Dual System for construction being estimated at approximately $293 000 000, and for equipment approximately $70 000 000.

Before the connection between the present four-track subway operating under Park Avenue and the new four-track subway under Lexington Avenue was definitely located, careful studies of a number of other locations were made, notably those at Union Square, at 33d Street, and at 40th Street. Notwithstanding the fact that the construction and real-estate cost of the 40th Street location was estimated to be less than that for any of the others, it was more objectionable from an operating standpoint, largely on account of the sharp reverse curves between stations. The connection extending from Park Avenue below 40th Street to Lexington Avenue at 43d Street was finally selected as being the best, all things considered. It has become locally known as the "Diagonal Connection" because it crosses diagonally the site of the old Grand Union Hotel south of 42d Street and the New York Central Railroad property north of that street. The new Grand Central subway station is on this diagonal line. The curves into Park and Lexington Avenues are placed, with reference to the station, so that the trains are either slowing down for the stop or gathering headway, and therefore no reduction of speed is required on account of these curves.

The construction of the connection between the old and new subways was made extremely difficult by the necessity of maintaining the constant operation of trains on perhaps the most intensively operated railroad in the world. In rush hours a train passes on one of the four tracks every 30 sec., and the interval between the ten-car express trains at that time is 1 min. and 48 sec. The subway now carries about 1 400 000 passengers daily, of whom probably 1 000 000 pass the work described in the paper. The head-room over the trains is limited, and there is little side clearance. For these and other reasons the work had to be carried on outside of the present structure. It was necessary, however, to occupy the tracks for certain operations, such as the placing of the columns and girders which are inside of the old subway struc-

* New York City.

ture. This work was done in the early morning hours, when the express trains were not running, and the local trains were operated on a 7-min. headway, as stated by the author. Mr. Ridgway.

The two north-bound tracks of the old structure, where the new work passes under it, were originally in a concrete-lined tunnel. This tunnel has been changed into a steel bridge, without interrupting train operation. In a number of instances the columns supporting the new tracks were set in shafts excavated in the rock, some of these shafts being stoped up from the new tunnel below. The difficulties are little realized except by those who have an intimate knowledge of the work. Certainly, the million passengers who were carried safely every day had little conception of what was going on about them. There is an old saying that accidents do not occur where they are expected; that is, because the dangers are realized and measures are taken to guard against them. That there was no accident of a serious nature indicates an intelligent understanding of the problems involved, and it speaks well for the careful attention to the details of the work by the engineers and contractors, and for the methods which they devised and followed. Much credit is due to the contractor's organization under the direction of Robert A. Shailer, M. Am. Soc. C. E., Tunnel Engineer for the Rapid Transit Subway Construction Company, and particular mention should be made of Mr. Perrine, the author of this paper, and of the General Superintendent, Thomas McCormick, who inaugurated and worked out many of the ingenious methods which were used so successfully in the work. The speaker would also add a word of appreciation of the services of the men who designed the work, of the Division Engineer in charge for the Public Service Commission, John H. Myers, Assoc. M. Am. Soc. C. E., and of his two assistant engineers, Mr. Stephen Schmidt and Melville S. Miller, Assoc. M. Am. Soc. C. E., who deserve the highest commendation. The fact that the work has been successfully carried to its present state of practical completion and that everything has come together so well indicates, not only the watchfulness and ability of the field engineers and contractors, but the excellent co-operation between them. Of course, honest differences of opinion occurred, and there were many discussions as to whether a detail of the work should be done in this way or in that way, but there was good team work, and the speaker believes the engineers and contractors have a wholesome respect for each other. In fact, without this team work he is sure the results would have been far less satisfactory.

Many will remember the tragic history connected with the construction of the present Park Avenue subway tunnels. There was an explosion of a magazine in front of the Murray Hill Hotel in 1902, which caused much loss of life and damage to property. Following

Mr. Ridgway.

this a rock slide occurred in the easterly one of the two tunnels, which carried with it the fronts of several buildings on the east side of Park Avenue between 37th and 38th Streets, and, later, the contractor, the late Ira A. Shaler, M. Am. Soc. C. E., was mortally injured by a fall of rock from the roof of one of the tunnels, an accident which terminated the promising career of an able engineer who had accomplished great things.

When demolishing an old structure, it is always a satisfaction to find that it was honestly built, and this may be said of the concrete lining of the Park Avenue tunnels previously mentioned, which was cut out in connection with the new construction. In order to test its strength, two samples were cut out and sawed into 6-in. cubes. In taking the samples, an attempt was made to select concrete which appeared to be of a quality not as good as the average in the structure. When these samples were tested, one developed a compressive strength of 3 940 lb. per sq. in., the other did not break at the capacity of the machine, equivalent to about 4 250 lb. per sq. in. This concrete was about 13 years old. The timber which was used in temporarily supporting the rock in the old tunnels, and which was then built into the concrete, was very sound, and some of it was used in the new construction for temporary blocking.

The author mentioned the fact that the lattice girders taken from the Second Avenue Elevated Railroad in connection with the third-tracking of that structure were used for temporarily supporting the street decking. It is interesting to note that these girders had been in use in the elevated structure for more than 35 years, and were in good enough condition to carry the new burden placed on them. In view of the present difficulty of getting new steel, it is likely that further use will be found for them.

Mr. Quimby.

HENRY H. QUIMBY,* M. AM. SOC. C. E.—Although this paper makes its special appeal to the contracting and field branches of the Profession, it is of very great interest to the designing engineer also, for a knowledge of the latest and most scientific methods of construction is necessary to the making of intelligent, feasible, and successful plans.

All who had the privilege of visiting the work during its progress were much impressed with the serious character of the contractor's task, and the ingenuity with which the construction problems were solved. The work looked ticklish, and evidently required much more than ordinary care, in order to avoid disaster. The visitor could not fail to appreciate, also, the ingenuity of the designers of the structure—both as a whole and in detail—with its successful solution of the perplexing questions imposed by the routing plans of the new transit lines; thus the work is a remarkable achievement, both on

* Philadelphia, Pa.

the part of the engineers of the Public Service Commission and on the part of the contractor's engineers. Mr. Quimby.

A noticeable feature of the paper is the evidence it contains of the cordial spirit of co-operation that prevailed between the contractor and the administrators of the contract. The fact is shown also in Mr. Myers' contribution to the subject, and is emphasized in Mr. Ridgway's discussion; all of which confirms the theory that the owner's engineer should regard the contractor, not as an enemy to be contended against, but as an associate to be co-operated with, for both desire the same thing—a first-class job, with safe and speedy completion.

The opportunity afforded by the demolition of an old engineering structure to learn something of the real results of construction methods is always appreciated, and, in the case of the old tunnel in Park Avenue, it was gratifying to see both the evidence of thoroughness in the efforts of its builders, and the fact of the complete success of the grouting operation—more gratifying than the revelations of demolition sometimes are.

The paper states that the contract provided for water-proofing above the arch over the station, but, after excavation had advanced, it was decided to omit it, and the arch was grouted and water-proofed inside. Apparently, the substituted method was not entirely successful, for work that has been going on there in cutting grooves, embedding drainage pipes, and plastering, seems to have been stopped, although there is still considerable leakage. What method of water-proofing above the arch was contemplated in the contract, and why was it abandoned; and is still another effort to be made to reduce the seepage?

C. V. V. Powers,* M. Am. Soc. C. E.—In regard to Mr. Quimby's question concerning the change in plan by which the water-proofing (six-ply fabric laid in hot coal-tar pitch) to be applied to the extrados of the arch was omitted, it appears to the speaker that the answer is very simple—it was not feasible to put in such water-proofing. Mr. Powers.

In 1901 or 1902, in the early days of the construction of the first New York Subway, a short section of tunnel arch was water-proofed with three-ply felt laid in hot asphalt, which experiment came directly under the speaker's observation. This trial section was on Broadway near 157th Street, at, and for a few feet north of, the south portal of the "Fort George Tunnel", the subway being a two-track structure. According to the plans, the concrete of the side-walls and of the arch for a height of 18 in. above the springing line was carried solid to rock; above that point the space between the extrados of the arch and the rock was to be filled with dry stone packing.

* New York City.

Mr. Powers.

The centers being set for concreting a section of arch, about 24 ft. long, the concrete of the arch itself was carried up for a height of about 2½ or 3 ft. on each side and for the full length of the section under construction, using back-forms to conform to the extrados of the arch. Work was then discontinued long enough to allow this concrete to set sufficiently to permit the back-forms to be removed. The surface, where necessary, was then smoothed over with cement mortar, which was allowed to dry. After this the water-proofers took their turn, laying the 3-ply water-proofing in hot asphalt on the portion of the arch just concreted, and for the length of the 24-ft. section, leaving also loose laps—about 8 to 12 in. in width—to be carried under the next lift of water-proofing. Incidentally, it was difficult to protect these laps from injury by the men working over them on the subsequent operations.

The back-forms were then set up again so as to provide for the 3 or 4 in. of "protective" concrete to be placed over the water-proofing, and, therefore, another delay occurred while this concrete was setting and becoming hard enough to resist damage during the placing of the stone packing. After this the stone packing was placed carefully by hand and carried as high as possible.

The foregoing operation was repeated again and again, carrying the concrete, water-proofing, etc., up step by step in heights of some 2½ to 3 ft., until the arch was completed. The height of each individual operation varied according to the room available for the men laying the dry packing to work in, but it is the speaker's recollection that, at the greatest, it would not average more than 3 ft.

As there was an open portal at one end of the 24-ft. section, and the arch had not yet been constructed at the other end, it was possible, though difficult, to water-proof the key of the arch. If both ends had been closed, the speaker knows of no way in which the arch could have been completely water-proofed.

The whole operation of concreting and water-proofing this 24-ft. section of arch, from an elevation 18 in. above the springing line, took—to the best of the speaker's recollection—almost exactly 1 month (due to the necessary delays mentioned and the difficulty of working in cramped quarters), and the operations were carried on under conditions much more favorable than the average, owing to the fact that the section under construction was at a tunnel portal, where there was more room to work and where, especially, there was a good supply of fresh air to diminish the danger from the suffocating fumes of the hot asphalt.

The final decision was very quickly reached, namely, that the result obtained was in no way commensurate with the great expense in time and money, and, further, that it was not feasible for men to work in a tunnel on the water-proofing of an arch, using hot

asphalt or coal-tar pitch. The speaker has been unable to find the detailed figures relative to the cost in time and money of doing this piece of work, but it was very high and was considered prohibitive. The experiment was not tried again on subway work, as far as the speaker knows. Mr. Powers.

It is believed that an informal attempt, on a small scale, was made to water-proof a portion of a tunnel arch, using felt or fabric and a cold water-proofing compound, but this did not give satisfactory results. The water-proofing compound, to be satisfactory, must not only be liquid when cold, but must harden quickly on application and still possess ductility and elasticity to a considerable degree. The speaker does not know of a satisfactory water-proofing compound of this nature, and if there is such, its use would remove only one of the objections to the water-proofing of tunnel arches.

The speaker has had some experience in endeavoring to prevent leakage through tunnel arches by applying a finish on the inside face. Something can frequently be accomplished in that way, but the good results are largely due to a liberal application of "elbow grease" in troweling the surface. It would be far better to apply the same amount of energy on the extrados of the arch, making the concrete on that surface as dense as possible.

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Paper No. 1409

OBSTRUCTION OF BRIDGE PIERS TO THE FLOW OF WATER*

BY FLOYD A. NAGLER, JUN. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. A. J. WILEY, R. D. GOODRICH, E. W. LANE, MANSFIELD MERRIMAN, F. H. FRANKLAND, CHARLES EVAN FOWLER, ROBERT E. HORTON, DAVID A. MOLITOR, AND FLOYD A. NAGLER.

SYNOPSIS.

This paper presents the results of 256 experiments which were performed in a flume at the Argo Dam, Ann Arbor, Mich., during October-December, 1914.

The experiments were made on 34 different models of bridge piers, in order to determine the relative obstruction to the flow of water offered by piers of different designs.

The method of conducting the experiments is described, and the existing theories for this phenomenon are reviewed; however, in order to show the relative efficiencies of these piers, it was found necessary to discard these theories. The paper presents a unique and simple formula which can be applied to this problem, making possible the comparison of the different piers by the magnitude of their coefficients in this formula.

Tables I to VII† contain all the experimental data, good and bad.

* Presented at the meeting of September 5th, 1917.

† Tables I to VII are not printed with the paper, but have been filed in the Engineering Societies Library, where they may be consulted by any one interested.

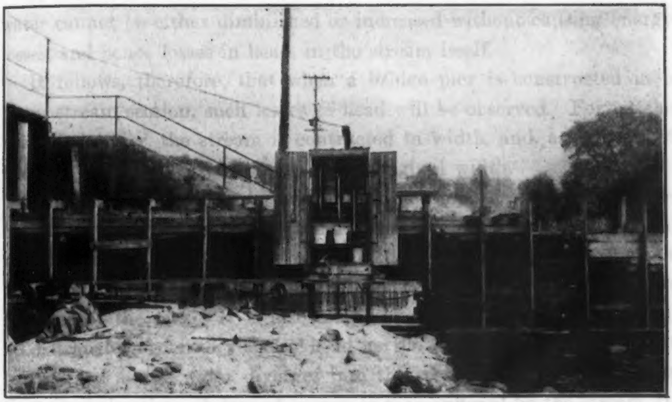


FIG. 1.—BOLTON'S FLUME.

FIG. 2.—BOLTON'S FLUME.

The Problem.

It is a well-known fact that the cross-sectional area of a stream of water cannot be either diminished or increased without causing energy losses, and hence losses in head, in the stream itself.

It follows, therefore, that when a bridge pier is constructed in a given stream section, such losses of head will be observed. For, at the nose of the pier, the stream is contracted in width, and, at the tail of the pier, it is suddenly enlarged to its original width.

Static head must first be available to accomplish the necessary increase in velocity demanded by the decrease in the stream section at the nose of the pier. Hence, there is observed the common phenomenon of "back-water", or the increase in the elevation of the water surface at the nose of the pier. It is also a well-known fact that all the static head, transformed into velocity head as the section converges, cannot be regained as the water diverges into the section of normal width at the tail of the pier. A loss of energy is thus suffered by the stream, in passing the bridge pier.

The losses, for convenience of these tests, are classified as follows:

(1) Losses which may be attributed to the change in section: These are made up of both surface and submerged losses, which are revealed by the turbulence and eddies at the nose and tail of the pier.

(2) Losses due to the friction of the water as it passes the wetted pier surface: These are relatively small when compared with those due to the change in section; they are so small, in fact, that, in most computations, they may be neglected.

It is at once apparent, on inspection of Fig. 8, that the shape of the nose and tail of the pier will have an appreciable effect on the magnitude of these losses. The pier, which will deflect the water at the nose, tangent to the sides of the pier, thus reducing to a minimum any further contraction of the stream section, should be most efficient at the up-stream end; and the pier with a tail which follows the diverging stream lines most closely, reducing the volume of the eddying water to a minimum, should be most efficient at the down-stream end.

Experimental data on the obstruction of bridge piers to the flow of water are exceedingly scarce; so scarce, in fact, that, in the case of the New York, Lackawanna and Western Railway *vs.* the New York,

Lake Erie and Western Railway,* engineers varied in their computations of the back-water caused by the proposed bridge piers of the Lackawanna Railroad in the Chemung River, by values ranging from 0.568 to 4.3 ft. Although it is true that part of this discrepancy was due to the different values of flood discharge assumed by the different engineers, yet most of it is attributable to uncertainty as to existing bridge pier formulas. The back-water caused by the proposed piers was of prime importance in deciding this case, because the piers and embankments of the Lackawanna Railroad, in crossing the Chemung flats, might raise the water of the Chemung River above the grade of the Erie tracks in flood stages of the stream.

Purpose of the Tests.

The purpose of these experiments was to determine the relative obstruction which different designs of bridge piers offer to the flow of water. This involved the derivation of an adequate formula, with reliable coefficients, which would give the amount of back-water caused by the insertion of a pier in the cross-section of the experimental flume.

Description of the Flume.

The tests were made in a flume constructed at the Argo Spillway, Ann Arbor, Mich. A head of 13 ft. of water was available for the experiments. This flume was constructed by Frank L. Bolton, Jun. Am. Soc. C. E., and was used by him in making extensive experiments on the efficiency of trash racks of different designs.

Figs. 1 and 2 are reproduced from photographs of the flume taken in October, 1914. Fig. 3 is a plan and section of the flume.

The sides of the flume were made of 2-in. matched yellow pine; the flume averaged 2.138 ft. in width, and could carry a depth of water of 4 ft. The quantity of water entering the flume was regulated by lowering or raising the flood-gate in the spillway. As the water entered the flume at a very high velocity, it was first necessary to lower this velocity and distribute the varying velocities uniformly over the entire section of the flume. This was accomplished by a set of baffles, shown on Fig. 3, as Baffles No. 2. Each set of baffles consisted of a set of vertical bars and a set of horizontal bars about 5 in. apart. Fig. 4 shows that an excellent distribution of velocity was obtained, in this

* "On the Determination of the Flood Discharge of Rivers and of the Backwater Caused by Contractions", by William R. Hutton, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 211.

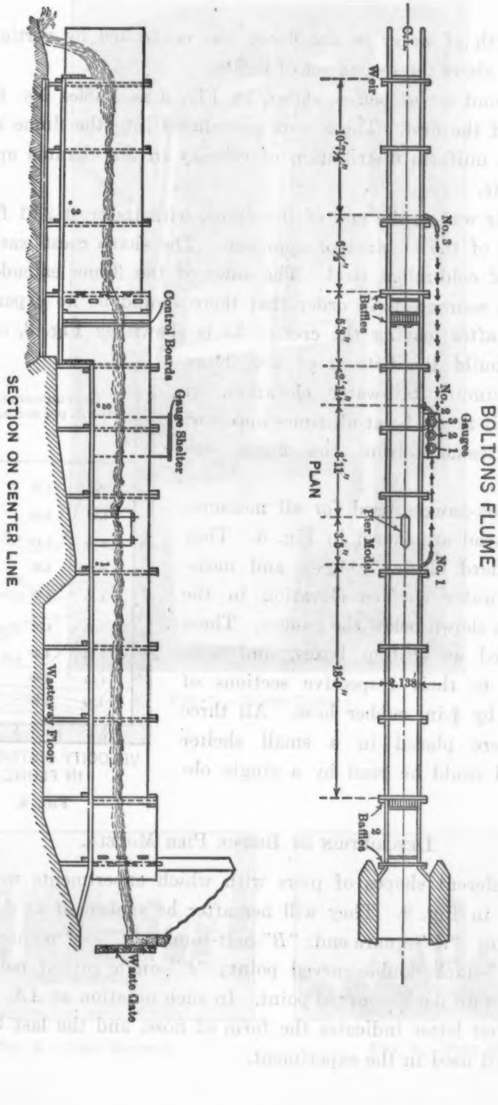

BOLTONS FLUME
PLAN
Weir
No. 3
No. 2
No. 1
Gauges
Baffles
Pier Model
Control Boards
Gauge Shelter
Waterway Floor
Waste Gate
SECTION ON CENTER LINE

Fig. 3.

manner, at a section 15 ft. below the baffles and 6 ft. above the testing section.

The depth of water in the flume was controlled by vertical stop-boards just above the second set of baffles.

The second set of baffles, shown on Fig. 3 as Baffles No. 1, was a duplicate of the first. These were introduced into the flume in order to secure a uniform distribution of velocity in the channel approaching the weir.

The weir was at the end of the flume, with its crest 2.01 ft. above the bottom of the channel of approach. The sharp crest was a 3 by ¼-in. bar of cold-rolled steel. The sides of the flume extended 4 ft. beyond the weir crest, in order that there should be no expansion of the nappe after leaving the crest. As is shown by Fig. 3, care was taken to build the bottom of the flume above maximum tail-water elevation, in order that it might be at all times apparent that the leakage from the flume was normal.

The hook-gauges used for all measurements of head are shown in Fig. 5. They were standard Gurley gauges, and measured the water surface elevation in the 12-qt. pails shown below the gauges. These pails served as stilling boxes, and were connected to their respective sections of the flume by ¾-in. rubber hose. All three gauges were placed in a small shelter house, and could be read by a single observer.

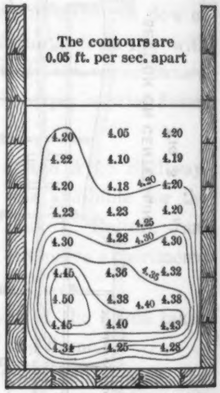


FIG. 4.

Description of Bridge Pier Models.

The different shapes of piers with which experiments were made are shown in Fig. 8. They will hereafter be spoken of as designated on the plate: *"A"*-square end; *"B"*-half-round; *"C"*-90° point; *"D"*-45° point; *"E"*-thick double-curved point; *"F"*-single-curved point (convex); *"G"*-thin double-curved point. In such notation as *AA, BC, GD,* etc., the first letter indicates the form of nose, and the last letter the form of tail used in the experiment.

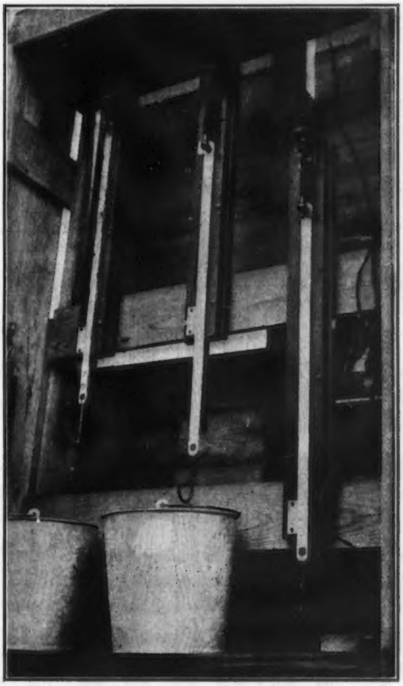


FIG. 5.—HOOK-GAUGES.

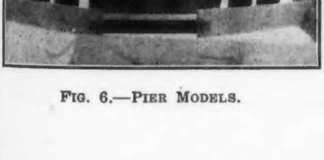


FIG. 6.—PIER MODELS.

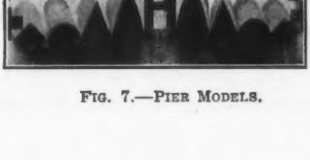


FIG. 7.—PIER MODELS.

Fig. 8.

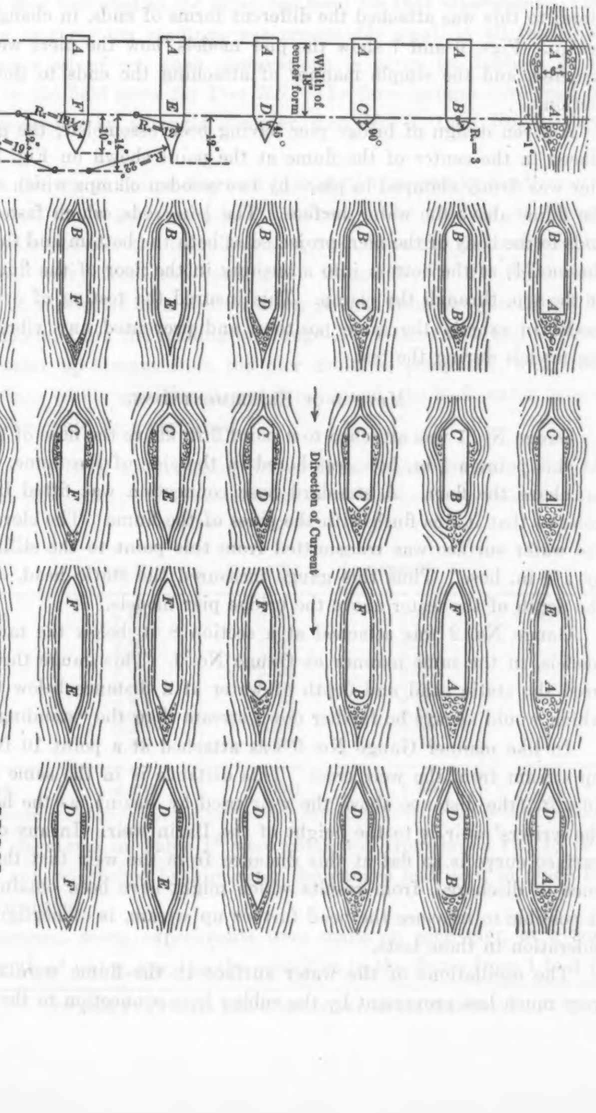


FIG. 8.

BRIDGE PIER MODELS

The models were of planed white pine. Each was 36 in. deep and 6 in. wide, although the gross length varied somewhat, according to the different designs. A center section 18 in. long was used for all piers; to this was attached the different forms of ends, in changing the design. Figs. 6 and 7 show the pier models, how the piers were constructed, and the simple manner of attaching the ends to the center section.

A given design of bridge pier having been assembled; the pier was placed in the center of the flume at the point shown on Fig. 3. The pier was firmly clamped in place by two wooden clamps which spanned the flume above the water surface. The ½-in. rods, which fastened the ends to the body of the pier, projected at both the bottom and the top of the model; at the bottom into a bushing in the floor of the flume; and at the top, through the clamp. This insured the testing of every pier model in exactly the same position, and prevented any vibration of the models during the test.

Method of Experimentation.

Gauge No. 1 was attached to a point 3 ft. above the nose of the pier. At this point a ¾-in. hole was bored in the side of the flume, about 2 in. above the floor. A standard hose connection was fitted into this hole, so that it was flush with the sides of the flume. The elevation of the water surface was transmitted from this point to the stilling pail by a ¾-in. hose. Thus this gauge measured the static head, and also the depth of the water above the bridge pier models.

Gauge No. 2 was attached at a section 8 ft. below the tail of the models, in the same manner as Gauge No. 1. This gauge then measured the static head and depth of water at a distance below the pier which would always be farther down stream than the "standing wave."

In like manner Gauge No. 3 was attached at a point 10 ft. 7½ in. up stream from the weir crest. This distance is in the same ratio to 16.4 ft. (the distance above the weir used by Bazin), as the height of the writers' weir is to the height of the Bazin weir. In any case, the surface curve is so flat at this distance from the weir that the difference in discharge, from results which might have been obtained were it possible to measure the head farther up stream, is of negligible consideration in these tests.

The oscillations of the water surface in the flume were rendered very much less prominent by the rubber hose connection to the stilling

wells. The water surface could easily be read to 0.001 ft. with the hook-gauges, and in many experiments to 0.0005 ft.

The arithmetical mean of ten observations at a given velocity and depth of flow was used as the measured head for that experiment. The variation in these ten readings was remarkably small in all the runs. An average set of ten such observations is given in Table 1 as it appears in the field notes for Pier *BB*. The three gauges were read in rapid succession by a single observer, first No. 1, then No. 2, then No. 3.

From the ten gauge readings all information necessary for the computation of the experiments was obtained. The reading of Gauge No. 3 made possible the calculation of the discharge, as measured by the Bazin weir; from the reading of Gauge No. 2 the depth and velocity of the water at a section down stream from the pier could be obtained; and from the reading of Gauge No. 3 the depth and velocity of the water up stream from the pier could be computed; and from the readings of Gauges Nos. 1 and 2 combined, the back-water caused by the pier could be ascertained.

TABLE 1.—AVERAGE SET OF TEN OBSERVATIONS.

Experiment.	Readings of Gauges, in Feet.			Time.
	Gauge No. 1.	Gauge No. 2.	Gauge No. 3.	
BB.5	0.871 0.872 0.870 0.870 0.873 0.873 0.873 0.873 0.870 0.870	0.833 0.830 0.829 0.831 0.833 0.832 0.833 0.833 0.829 0.828	0.395 0.396 0.395 0.396 0.397 0.398 0.397 0.398 0.397 0.396	9.13 A. M.
Mean.	0.8715	0.8311	0.3960	

The elevations in Table I* were computed from levels run by Mr. Bolton, and re-checked by him after the gauges had been in place for several months.

In general, seven experiments were made on each pier, at a constant depth of water, varying the velocities in the flume from 1 to 4 ft.

* Tables I to VII are filed in the Library of the Society.

per sec. In addition to this, on Piers *AA, BD,* and *DB,* the depth of the water was varied for constant velocities. The different experiments on a single pier are designated in Tables I to VII by the numbers, 1, 2, 3, etc.

Computations.

The discharge for a given run was read from the curves on Fig. 9. The upper curve gives the discharge of the weir as determined by Bazin's formula for a weir with a crest 2 ft. above the bottom of the approach channel. The lower curve was computed from the upper curve, by increasing the discharge 1% in order to compensate for the leakage of the flume between the weir and the bridge piers; thus, it gives the discharge at the bridge pier section.

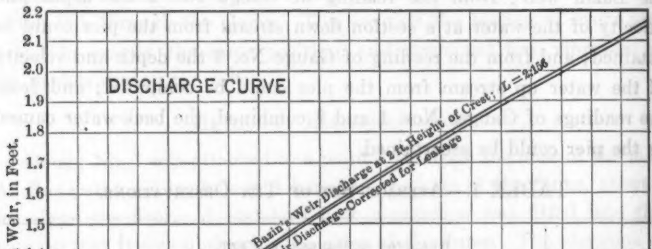


Fig. 9.

The foregoing leakage correction was determined jointly with Mr. Bolton, by catching the leakage, in a definite period of time, from several of the 3-ft. sections of the flume. This was also checked by measurements with a small Ott current meter at the pier section, and also just above the weir.

The width of the weir crest, and also the average width of the flume above it was 2.156 ft. The mean width of the flume at Gauges Nos. 1 and 2 was 2.138 ft.

Due to the limitations in the accuracy of the weir formulas, and also the fact that the experiments with the higher velocities gave heads as high as 2.1 ft. on the weir, it has been concluded that the absolute error in these tests may be as great as 3%, and the relative error as great as 2 per cent.

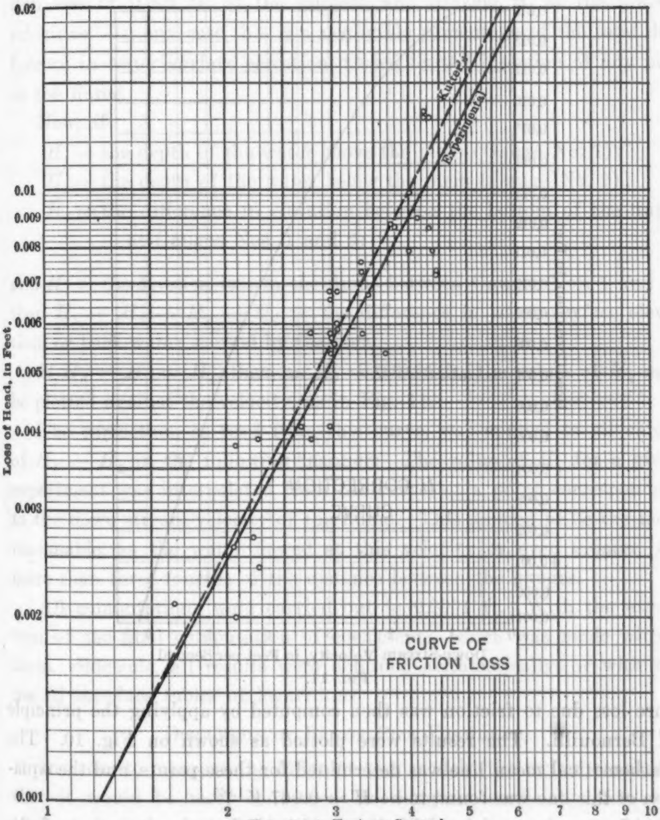


FIG. 10.

In all computations the pure friction losses of the pier models were neglected. This seemed to be permissible, as this loss amounts to less than 0.001 ft. for the highest velocity recorded in these tests. It was

thus possible to make comparisons between piers differing in gross length due to changing the form of the nose or tail.

Mr. Bolton determined the friction losses of the flume itself, between Gauges Nos. 1 and 2, by a series of thirty-three experiments at different velocities and depths, with no obstruction in the flume. The

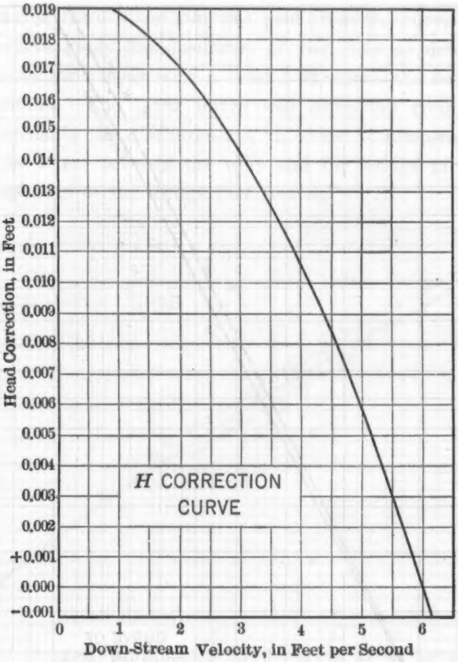


FIG. 11.

pure loss due to friction was then computed by applying the principle of Bernouilli. The results were plotted as shown on Fig. 10. The mathematical mean line was determined for these points, and the equation of this line was found to be $H = 0.067\ V^{1.866}$.

After very careful study, it was found that the variation of the depth of the water in the flume, within the range of the experiments performed, seemed to produce so little variation in the value of the friction loss, that this loss in the flume for the varying velocities was read from the mathematical mean line, regardless of the depth of

water in the flume. The close proximity of this line to that obtained from Kutter's coefficient for a wooden channel, with water 3 ft. deep, is an excellent check on the accuracy of these experiments. A variation in depth of 0.5 ft. will cause a variation in the value of the friction loss of 0.001 ft. at the highest, and 0.00005 ft. at the lowest, velocities. In any case this is a negligible percentage of the total difference in water surface elevation caused by the insertion of any pier in the flume.

Now, if

H_1 = the depth of the water above the piers (Gauge No. 1);

H_2 = the depth of the water below the piers (Gauge No. 2);

H_d = the difference in elevation between the bottom of the flume at Gauges Nos. 1 and 2;

and H_f = the friction loss in the flume between Gauges Nos. 1 and 2; then $H_1 + H_d - H_2 - H_f$ = the difference in water surface elevation, or back-water, caused by the pier.

If $H_d - H_f = H_4$, then, as $H_d = 0.0195$ ft., the values of H_4 may be plotted against the velocity, as in Fig. 11.

The correction, as read from this curve, was applied to all values of $H_1 - H_2$ in the following manner: The value of H_4 for a given experiment was interpolated from the curve, at a point corresponding to the down-stream velocity of the flume. This seemed to be the most reasonable, as the water flowed at this velocity or even greater, for more than three-fourths of the distance between the gauges.

All computations were carried out to 0.0001 ft. With the exception of the final computation of coefficients, which was made with a 10-in. slide-rule, all results were obtained arithmetically, or with the use of six-place tables of logarithms.

Discussion of Bridge Pier Formulas.

In order to compare the different pier models, the writer concluded that it would be most convenient to compute coefficients for each kind of pier, these coefficients to be applied to some adequate bridge pier formula. It became necessary, therefore, to study the existing formulas for back-water caused by bridge piers.

D'Aubuisson, Eytelwein, Debauve, Dupuit, and Gauthey were among the first to investigate this phenomenon. In general, the theory adopted by these investigators was to attribute the back-water to the

velocity change as it entered the contracted section of the stream, in all cases applying a coefficient of contraction.

It was first developed by D'Aubuisson, although commonly attributed to Dubuit or Debauve, as follows:

Let V_0 = the velocity in the approach channel;
V_2 = the velocity between the piers;
W = the full width of the approach channel;
w = the contracted width at the pier section;
h = the normal depth of the stream below the piers;
y = the amount of back-water; and
m = the coefficient of contraction.

Then, as
$$V_2 = \frac{V_0 W}{w}$$
$$y = \frac{V_2^2 - V_0^2}{2g} = \frac{V_0^2}{2g}\left(\frac{W^2}{w^2} - 1\right).$$

Or,
$$y = \frac{V_0^2}{m^2 2g}\left(\frac{W^2}{w^2} - 1\right).$$

Eytelwein, however, pointed out that V_2 is not equal to $\frac{V_0 W}{w}$, because, due to the crest contraction, the depth of the water between the piers is less than the depth ahead of them. He also argued that the coefficient of contraction should only be applied to the contracted width. He thus modified the foregoing formula until it assumed the form:
$$y = \frac{V_0^2}{2g}\left(\frac{W^2}{m^2 w^2} - \frac{h^2}{(h+y)^2}\right)$$

This, in all probability, is the most correct formula which has been derived, taking the preceding theory as a basis.

Debauve expresses the same formula in terms of the discharge of the stream, Q.
$$y = \frac{Q^2}{2g}\left(\frac{1}{m^2 w^2 h^2} - \frac{1}{W^2 (h+y)^2}\right).$$

Gauthey gives the following values for m in this formula:

0.95 for acute angles,
0.90 for half-rounded heads,
0.70 for blunt, or square heads.

The preceding formulas have been disregarded by modern investigators, as the losses at the tail of the pier and the standing wave have

been entirely neglected. This would tend to make y less than its theoretical value, if observed at the head of the enlargement, where y is a maximum. The losses at the tail of the pier are considered by many investigators to exceed those of contraction at the head of the pier. Furthermore, the results of these experiments, if substituted in the foregoing formulas, give a regularly decreasing variation in the value of m for a single pier, within the range of from 1 to 4 ft. velocity, of more than 9 per cent. The values of m, computed from these tests, all lie between 1.07 and 1.28, and are thus much higher than those given by Gauthey. Certainly, in any case, a formula which requires a table of coefficients varying through a range of 9% is little better than no formula at all.

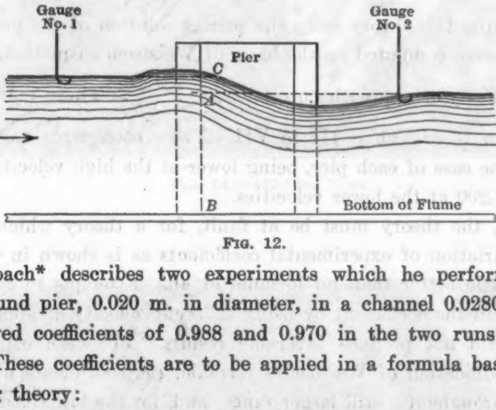


FIG. 12.

Weisbach* describes two experiments which he performed on a small round pier, 0.020 m. in diameter, in a channel 0.0280 m. wide. He derived coefficients of 0.988 and 0.970 in the two runs which he made. These coefficients are to be applied in a formula based on the following theory:

In Fig. 12 if H is equal to the difference in elevation between Gauges Nos. 1 and 2, he considers the discharge through the pier section to be equivalent to that through an orifice of a section, AB, and under a head, H, and over a weir the length of which is the net stream width at A, under a head, H. The sum of these two discharges should give the discharge past the pier.

Thus, if W is equal to the width of the channel between the piers, and D_2 is equal to the depth of the water at Gauge No. 2, then

$$Q = C\,W\sqrt{2g}\left(\frac{2}{3}H^{\frac{3}{2}} + D_2 H^{\frac{1}{2}}\right).$$

* "Experimental Hydraulik."

Mansfield Merriman, M. Am. Soc. C. E., adopts Weisbach's theory,* with some modification. In the first place, he corrects H for the velocity in the approach channel, by adding to the measured difference in elevation, H, a quantity equal to $1\,\frac{V_1^2}{2\,g}$, where V_1 is the velocity of the stream. In the second place, he considers the weir portion of the theoretical discharge to be continuous over the entire width of the stream, and that the orifice discharges through the contracted section, only. Thus, if B is equal to the width of the approach channel, and b is the width at the pier section, then:

$$Q' = c\,\sqrt{2\,g}\left[\frac{2}{3}\,B\left(H+\frac{V_1^2}{2\,g}\right)^{\frac{3}{2}} + b\,D_2\left(H+\frac{V_1^2}{2\,g}\right)^{\frac{1}{2}}\right]$$

Assuming this theory to be the correct solution of the problem, coefficients were computed on the basis of Weisbach's equation, applying to H a velocity of approach correction of $1\,\frac{V_1^2}{2\,g}$. These may be found in Column 16 of Tables III to VII. These coefficients vary at least 20% in the case of each pier, being lower at the high velocities and as large as 1.200 at the lower velocities.

Again, the theory must be at fault, for a theory which gives as wide a variation of experimental coefficients as is shown in the tables is very little better than no formula at all. Attempts to reduce this variation in the coefficient by using different velocity of approach corrections did not produce favorable results. Likewise, using Merriman's modification of Weisbach's formula, the coefficients were found to vary throughout a still larger range, and, for the low velocities, they are practically equal to those tabulated in Column 16 of Tables III to VII. Merriman's assumption, that the weir discharges over the entire width of the stream, seemed to make conditions worse, rather than better. Certainly, there is little if any argument for assuming that the weir discharges over one section, and the orifice at another section, of the stream, and then adding the two to obtain the total discharge. A very fundamental principle of hydraulics is violated in so doing. Fig. 13 is a copy of the figure used by Mr. Merriman in demonstrating his formula.† Certainly,

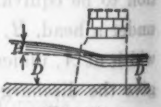


FIG. 13.

* "Treatise on Hydraulics."

† "Treatise on Hydraulics," Ninth ed., p. 343.

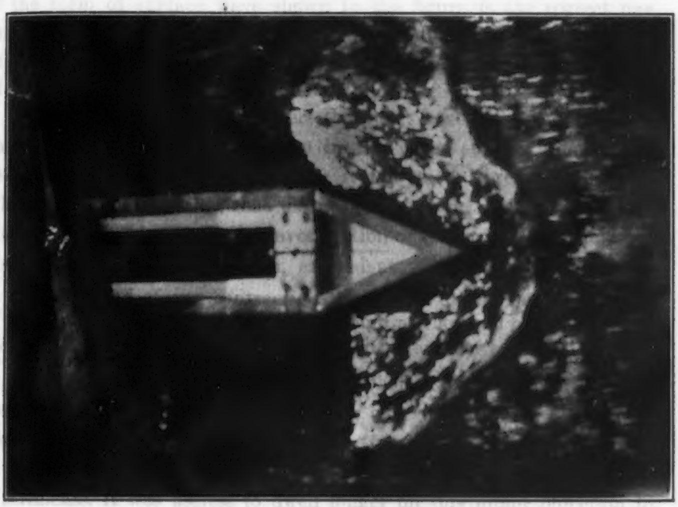


FIG. 14.—45° PIER NOSE.

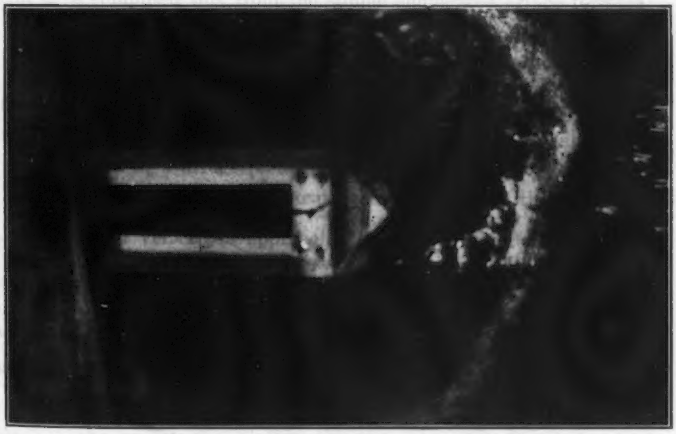
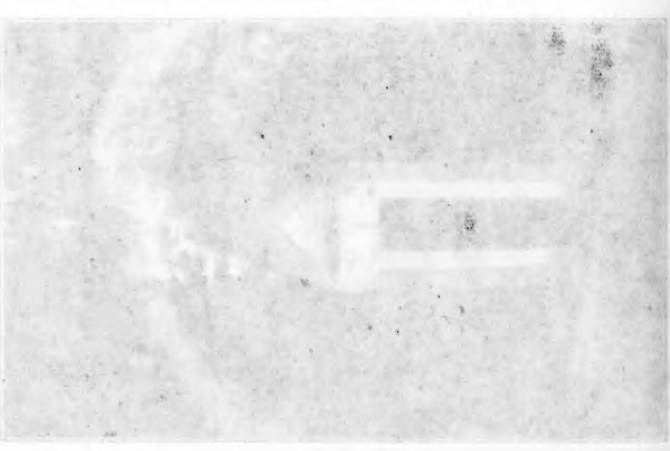


FIG. 15.—HALF-ROUND PIER NOSE.



if the form of surface wave shown in the figure is the correct one, and the one which Mr. Merriman saw when watching water pass bridge piers, there may be more ground for the assumption that the weir discharges over the entire stream width.

Weisbach, Gauthey, and others, however, have been very clear in their drawings in showing a rise in the water surface immediately ahead of the pier, and neither does it fall until it reaches the pier. This, certainly, is what has been observed in these experiments. Dupuit, also, in his theoretical investigation of the form of surface curve obtained by lateral contraction, arrived at the following conclusion:

"At the head of any contraction there must always be an elevation of the water surface above the normal; and at the head of any enlargement there must always be a depression of the water surface."

Figs. 14 and 15 are reproduced from photographs of the bridge pier models in the flume with water flowing at a velocity of 4 ft. per sec. They show clearly the standing wave in front of the pier.

As the theory as a whole, however, does not coincide with these experiments, it was useless to dwell longer on this minor objection to Mr. Merriman's formula; the main error lies elsewhere.

If the writer had been required to calculate the loss in head through a sluice-gate with the form of section shown in Fig. 16, he might have computed this from the simple submerged orifice formula:

$$Q = C\,A\,\sqrt{2\,g\,H}.$$

Neglecting the friction of the concrete, the discharge would not differ whether X were of concrete or of air. If of air, the form of the surface curve would be the same as that drawn in Fig. 12, and caused by the insertion of a bridge pier.

Furthermore, turning to the experimental data for the pier, AA, H, corrected for velocity of approach, $1\,\frac{V_1^2}{2\,g}$, was plotted as a function of $\frac{Q}{D_2}$ where D_2 is the down-stream depth of the water. The result is shown on Fig. 17. It was at once evident that $\frac{Q}{D} = f\,(H)$ was almost purely exponential, and that this exponent is approximately $\frac{1}{2}$. On investigation this was found to be true for the other piers also. These experiments thus seemed to indicate that the theory of considering the discharge past bridge piers as being that of a combined

weir and orifice is false; and, furthermore, that the discharge might more correctly be represented as that through an orifice of a sectional area equal to the minimum stream width multiplied by the minimum stream depth, and under a head, H, equal to the back-water caused by the bridge pier (corrected for velocity of approach).

This theory is not new. Both Unwin and Frizell seem to have arrived at a similar conclusion in determining the discharge over a broad-crested weir. They found that

$$Q = L\, d \sqrt{2\, g\, (H - d)},$$

in which L is the length of the weir; d is the depth of water on the weir; and H is the elevation of the water surface ahead of the weir, above the weir crest, as in Fig. 18.

As $(H - d)$ is the difference in water surface elevation caused by the weir, the discharge has been computed as being that through an orifice at section, B, and under a net head, h. The crest and all hydraulic conditions in this case are very similar to the crest (obtained by the writer) due to the contraction of the channel by a bridge pier. In fact, Dupuit, in his theoretical investigation of this problem, states that the same resulting surface curve follows whether the contraction is from the sides or bottom of the stream. There may be some basis, then, for the assumption that, if this theory has been applied successfully to broad-crested weirs, it may as well be applied to bridge pier sections, where the surface curve is the same.

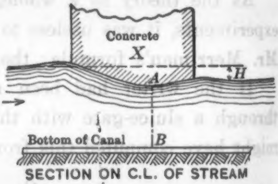


Fig. 16.

Bridge Pier Formula Used in the Reduction of These Experiments.

Referring to Fig. 19, the following base formula for discharge past a bridge pier—or back-water caused by bridge piers—was deduced as the result of these tests. In order to make comparison between the different pier models, the entire discharge in the stream is considered as passing through an orifice at the section, A.

If A = the area of this submerged orifice,

H = the head on that orifice, and

Q = the discharge through that orifice,

then $Q = C\, A \sqrt{2\, g\, H}$, in which C is an empirical coefficient.

Now, in this case, $A = WD$, where D equals the depth of water in Section A, and W equals the width of the stream, exclusive of the bridge piers. Thus

$$Q = C\, W\, D \sqrt{2\, g\, H}.$$

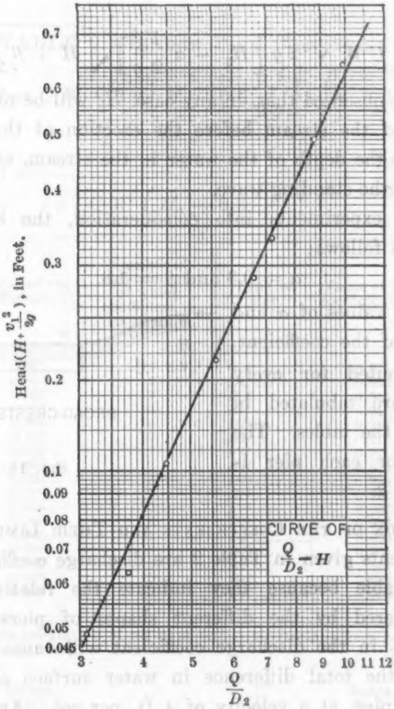


FIG. 17.

This is the base formula for these experiments, in its theoretical form, with no corrections. However, if the depth of water is measured at Gauge No. 2, designating this depth by H_2, then

$$D = H_2 - \alpha \frac{V_2^{\,2}}{2\, g},$$

where V_2 = velocity of retreat.

H must also be corrected for the velocity in the channel of approach, thus

$$H = H_a + \beta \frac{V_1^2}{2\,g}.$$

The formula, with its corrected values, then assumes the following form:

$$Q = C\,W \sqrt{2\,g}\left[H_2 - \alpha \frac{V_2^2}{2\,g}\right]\sqrt{H + \beta \frac{V_1^2}{2\,g}}.$$

It should be observed that, in any case, H_2 will be practically equal to the depth of the stream before the erection of the bridge piers, and is equal to the depth of the water in the stream, at a point down stream, beyond the standing wave.

Taking all experiments into consideration, the best values of α and β are as follows:

$$\alpha = 0.3 \text{ and } \beta = 1.8.$$

Using these values of α and β, the values of the coefficient, C, were computed for every run. These are tabulated in Column 19 of the tables. The mean value for each pier is given in Table 2.

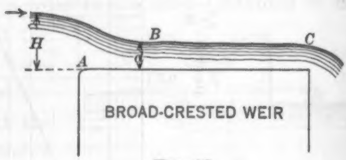


FIG. 18.

DISCUSSION OF THE COEFFICIENTS AND THEIR LIMITATIONS.

The coefficients given in Table 2 are discharge coefficients, and are primarily valuable because they indicate the relative amount of obstruction offered by the different shapes of piers; that is, an increase of 1% in the discharge coefficient will cause a decrease of about 7% in the total difference in water surface elevation above and below the pier, at a velocity of 4 ft. per sec. An inspection of the formula at once shows that the height of the back-water varies as the square of the velocity, also as the square of the discharge, and inversely as the square of the coefficient of contraction, all other values in the equation remaining the same.

A study of the coefficients obtained throughout the range of experiments on a single pier shows variations from the mean value of less than 1%, in most cases; there being only ten experiments where the variation of the coefficients computed for any single experiment is

greater than this. Far less variation than this could have been obtained by adopting different coefficients for the different pier shapes, but such a table of coefficients would be cumbersome, hence mean values of the coefficients were adopted ($\alpha = 0.3$ and $\beta = 1.8$) which gave the best results, considering the entire thirty-four different shapes of piers.

TABLE 2.—BRIDGE PIER COEFFICIENTS.*

$$Q = C W \sqrt{2g}\left[H_2 - 0.3\frac{V_2^2}{2g}\right]\sqrt{H + 1.8\frac{V_1^2}{2g}}$$

or,

$$H = \frac{Q^2}{2g\left[C W\left(H_2 - 0.3\frac{V_2^2}{2g}\right)\right]^2} - \frac{1.8\,V_1^2}{2g}.$$

Type of pier.	Value of the coefficient, C.	Number of experiments of which coefficient is a mean.	Type of pier.	Value of the coefficient, C.	Number of experiments of which coefficient is a mean.
AA........	0.861	16	*GA*........	0.911	4
AC........	0.862	7	*DC*........	0.912	7
AF........	0.863	7	*BA*........	0.914	7
AG........	0.863	6	*BC*........	0.915	7
AB........	0.866	6	*DD*........	0.916	7
AD........	0.873	7	*DB*........	0.918	15
AE........	0.875	7	*FC*........	0.921	7
CC........	0.893	7	*BB*........	0.923	7
CA........	0.894	6	*BD*........	0.923	17
CD........	0.902	7	*BG*........	0.927	6
FA........	0.903	7	*FB*........	0.927	7
CB........	0.905	6	*FD*........	0.927	7
CF........	0.905	7	*DF*........	0.929	7
CG........	0.906	6	*DE*........	0.931	7
CE........	0.907	7	*BF*........	0.931	7
EA........	0.908	5	*BE*........	0.935	7
DA........	0.910	7	*FE*........	0.939	7

*** A complete tabulation of all data and computations may be found in the Engineering Societies Library.**

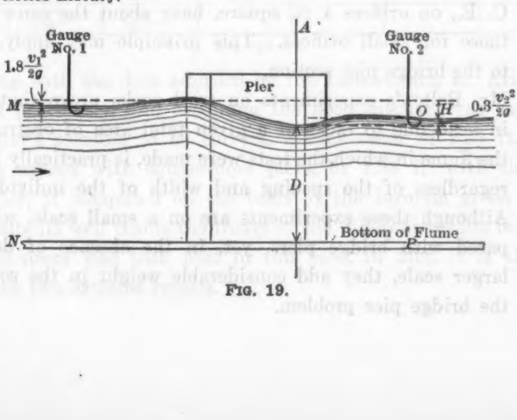


FIG. 19.

As the weir over which the discharge was measured was of limited capacity, these experiments cover a range in velocity of from 1 to 4 ft. per sec., only. Whether the coefficients for the formula will still have the same value for higher velocities is merely a matter of conjecture, although it would not be at all unreasonable to assume the same coefficients for slightly higher velocities of flow.

The formula is applicable to varying stream depths and hydraulic radii. This is shown by the coefficients computed for Piers *AA, BD,* and *DB,* on which tests were made through a very wide variation in the depth of the water in the flume.

As the velocity of retreat, for most problems, is only a very small amount greater than the velocity of approach, the computations may be somewhat simplified by assuming them to be of the same value; hence, if h is the head due to the velocity of approach, then

$$Q = C\,W \sqrt{2g}\,(H_2 - 0.3\,h) \sqrt{H + 1.8\,h}.$$

It has been claimed by some writers that the amount of back-water will vary with the relative velocities above and below the pier, the number of piers, the span of the bridge, the thickness of the piers, and the profile of the stream bottom.

The limitations in the size of the flume permitted tests to be made on only one particular width between piers. Whether these coefficients can be extended to other widths between piers is a matter of conjecture. The following are considerations which lead one to believe that this formula may be applicable to a stream of any width, and for any number of piers:

(1) Coefficients deduced by C. B. Stewart, Assoc. M. Am. Soc. C. E., on orifices 4 ft. square, have about the same value as those for small orifices. This principle may apply, as well, to the bridge pier section.

(2) Mr. Bolton's experiments on trash-racks prove that the loss in head due to racks of a given total area of obstruction, in the flume in which the tests were made, is practically the same, regardless of the spacing and width of the individual bars. Although these experiments are on a small scale, when compared with bridge piers, yet, in the absence of data on a larger scale, they add considerable weight in the solution of the bridge pier problem.

(3) The width of the stream is taken into consideration in this formula, in the large velocity of approach correction.

Weisbach's experiments, referred to herein, give values of coefficients which do not check with the values given here. His channel was so small, however, and his pier so nearly obstructed the entire flow of the water (leaving only 4 mm. on each side of the pier), that it does not seem at all unreasonable that his experiments should not check. It certainly has been a mistake to give these experiments as much prominence as they have received.

COEFFICIENTS

RELATIVE NOSE EFFICIENCY

"A" Tail	"B" Tail	"C" Tail	"D" Tail	"E" Tail	"F" Tail	"G" Tail
AA 0.861	AB 0.866	AC 0.862	AD 0.873	AE 0.875	AF 0.863	AG 0.863
CA 0.894	CB 0.905	CC 0.893	CD 0.902	CE 0.907	CF 0.905	CG 0.906
FA 0.903	DB 0.918	DC 0.912	DD 0.916	DE 0.931	DF 0.929	BG 0.927
EA 0.908	BB 0.923	BC 0.915	BD 0.923	BE 0.935	BF 0.931	
DA 0.910	FB 0.927	FC 0.921	FD 0.927	FE 0.939		
GA 0.911						
BA 0.914						

RELATIVE TAIL EFFICIENCY

"A" Nose	"B" Nose	"C" Nose	"D" Nose	"E" Nose	"F" Nose	"G" Nose
AA 0.861	BA 0.914	CC 0.893	DA 0.910	EA 0.908	FA 0.903	GA 0.911
AC 0.862	BC 0.915	CA 0.894	DC 0.912		FC 0.921	
AF 0.863	BB 0.923	CD 0.902	DD 0.916		FB 0.927	
AG 0.863	BD 0.923	CB 0.905	DB 0.918		FD 0.927	
AB 0.866	BG 0.927	CF 0.905	DF 0.929		FE 0.939	
AD 0.873	BF 0.931	CG 0.906	DE 0.931			
AE 0.875	BE 0.935	CE 0.907				

FIG. 20.

Working with the data afforded by the Lackawanna *vs.* Erie Case, mentioned previously, and assuming the flood discharge as computed from Kutter's formula to be correct, a back-water of 2.30 ft. would have been caused with square-nose piers, or 1.95 ft. with the most efficient pier, if computed on the basis of the formula given herein. These results lie well inside the limits of the different results computed by the engineers who took part in this case, in fact, it is almost a mean of the two extreme results.

Fig. 20 shows how the coefficients vary with a given pier nose and different tails, and also the variation with a given tail and different noses.

The coefficients have been computed to the third decimal place, although the accuracy of the experimental work certainly does not warrant such refinement. This was done, however, in order to distinguish between the different designs of piers which have almost the same efficiency. On the other hand, any variation in the resulting coefficients, from that which one would expect from a general consideration of the effect produced by a given head or tail, occurs only in the third decimal place. In these particular cases of seeming inconsistency, carrying the results to the third decimal place may have been straining the accuracy of the actual tests. The almost general coincidence of results throughout the whole thirty-four different designs, even when considering the values of the coefficients carried to the third decimal place, speaks well for the relative accuracy of the experiments, in spite of a few inconsistencies.

CONCLUSIONS.

These experiments have made the following contribution to our knowledge relating to the back-water caused by bridge piers:

(1) It has been demonstrated that the existing formulas for the discharge past bridge piers, and the back-water caused thereby, are erroneous.

(2) In working up the data provided by these experiments, a formula, which is substantiated by the results of 255 experiments, was developed. It is submitted as being far more adequate and simple than any of the existing formulas. Experimental data are not at hand, however, which will assure the accuracy of the formula, if applied to conditions differing widely from those in the writer's experimental flume.

(3) Coefficients for thirty-four different designs of bridge piers are submitted, for application in the writer's formula.

(4) The main value of these coefficients lies in the fact that they show the relative efficiencies of the different piers. This is shown in Fig. 20. The more remarkable conclusions may be summarized as follows:

I.—The best practical form of nose is either the half-round or the elliptical (of which Pier *F* is a modification). These shapes give much better efficiencies than the pointed nose.

II.—The best form of tail is the double curved form, like a fish tail (Pier *E*). The back-water is materially decreased by the adoption of such a tail. This form, however, is not practical; either the *D* or *B* modifications produce the second-best results. These are both practical shapes.

III.—The discharge coefficient may be increased from 1 to 3% by substituting an efficient tail for the square one. This means a decrease of from 7 to 25% in the amount of back-water at a stream velocity of 4 ft. per sec. This demonstrates clearly the importance of the shape of the tail of a pier.

IV.—The discharge coefficient can be increased from 5 to 6% by substituting a half-round nose for a square one. This means a decrease in the amount of back-water of from 35 to 45%, with a stream velocity of 4 ft. per sec.

V.—The decrease in the amount of back-water obtained by substituting a 90° point at the tail of the pier for the square end, is negligible; hence it does not warrant the extra expense of constructing such an end. The round tail is far superior to the 90° point, and is almost as efficient as the 45° pointed tail.

VI.—For piers of the same design, up stream and down stream, the half-round ends give the least amount of back-water.

ACKNOWLEDGMENTS.

For the facilities offered by the Argo Spillway and for the use of the water in the Argo Pond, the writer is indebted to the Eastern Michigan Edison Company and to Gardner S. Williams, M. Am. Soc. C. E., its Consulting Engineer. For the use of the flume and valuable advice and assistance, and also considerable experimental data, the writer wishes to thank Frank L. Bolton, Jun. Am. Soc. C. E. The writer also wishes to acknowledge his indebtedness to Horace W. King, M. Am. Soc. C. E., Professor of Hydraulic Engineering at the University of Michigan, whose suggestions were invaluable in the conduction of these experiments.

DISCUSSION

Mr. Wiley. A. J. WILEY,* M. AM. SOC. C. E. (by letter).—The formula proposed by the author appears to fit the conditions under which his experiments were made, but it seems to the writer that these conditions, in which the pier occupied nearly 25% of the channel, are not comparable to the case of the ordinary bridge pier. In the author's experiments, the standing wave around the nose of the pier, as shown by the photographs, extended across the entire channel, whereas, in the case of an ordinary bridge pier, which occupies a relatively small part of the channel, the standing wave would not materially affect the hydraulic conditions.

In the author's formula for back-water:

$$H = \frac{Q^2}{2\,g\left[C\,W\left(H_2 - 0.3\,\frac{V_2^{\,2}}{2\,g}\right)\right]^2} - \frac{1.8\,V_1^{\,2}}{2\,g} \qquad (1)$$

H = head caused by back-water,
C = a coefficient,
W = width between piers,
H_2 = normal depth below piers,
V_1 = velocity of approach, and,
V_2 = velocity of retreat, or normal velocity below piers.

In the formula it is evident that the expression,

$$W\left(H_2 - 0.3\,\frac{V_2^{\,2}}{2\,g}\right),$$

is the cross-section in the minimum section between the piers, so that the author's formula may be written,

$$H = \frac{Q^2}{2\,g\,(C\,A)^2} - \frac{1.8\,V_1^{\,2}}{2\,g} \qquad (2)$$

Also,

$$\frac{Q}{A} = V_0$$

where A is the area and V_0 the velocity at the contracted section between the piers.

The formula thus becomes:

$$H = \frac{V_0^{\,2}}{C\,2\,g} - \frac{1.8\,V_1^{\,2}}{2\,g} \qquad (3)$$

The author's coefficient, C, varies from 0.861, for a pier with both ends square, to 0.939 for one with nose and tail tapered. It appears, therefore, that H would be zero, and there would be no back-water effect unless the contraction caused by the pier was such that the

* Boise, Idaho.

square of the velocity between the pier would be 1.55 times the square of the velocity of approach for a square-ended pier, or 1.68 times for a taper-ended pier. Of course, there must be some back-water effect, no matter how slight the contraction caused by the pier, and, unless the writer is wrong in his interpretation of the author's formula, it cannot be of general application. Mr. Wiley.

In one case observed by the writer, a concrete-lined canal is crossed by a wagon bridge with square-ended piers, 8 in. thick, and 14.32 ft. between centers. The following are the values of the different elements in this case:

$Q = 472$ sec-ft., between adjacent piers,
$W = 13.65$
$H_2 = 7.82$
$V_2 = 4.22$

Levels taken for a distance of 100 ft. above and below the bridge, on each side of the canal, which was 87 ft. wide at the water surface, and, corrected for the normal fall in the water surface, gave a back-water head of 0.077 ft. Levels taken in the center immediately above and below the bridge gave a back-water head of 0.15 ft. The author's formula gives the back-water head for these conditions as — 0.10 ft.

It will be observed that the author's formula, as reduced to Equation (3):

$$H = \frac{V_0^2}{C\,2\,g} - \frac{1.8\,V_1^2}{2\,g}$$

is the same as that of D'Aubuisson given by the author as:

$$Y = \frac{V_2^2 - V_0^2}{2\,g},$$

or, changing the notation to conform with that used in the author's formula,

$$H = \frac{V_0^2 - V_1^2}{2\,g},$$

except for the application of the coefficient, 1.8, to the velocity head above the piers and the use of coefficient, C.

The writer is of the opinion that this formula, as modified by Eytelwein and quoted by the author, is an entirely rational expression for the back-water, and will fit ordinary conditions better than that proposed by the author.

Changing the modified D'Aubuisson-Eytelwein formula into the terms used in the author's formula, it is as follows:

$$H = \frac{V_1^2}{2\,g}\left(\frac{W_1^2}{C^2\,W^2} - \frac{H_2^2}{(H_2 + H)^2}\right)$$

in which W_1 is the width of approach.

Mr. Wiley. The application of this equation to the case of the piers in the concrete-lined canal previously mentioned, gives a back-water effect of 0.130, which is probably as close to the actual back-water as the average head of 0.077 ft., or the center measurement of 0.15 ft.

The author's method of determining the depth at the minimum section between the piers is to assume that it will be less than that below the piers by the amount of the recovered velocity head, which, of course, is correct; but this percentage of recovery would vary for different conditions, and would have to be assumed for any other condition than that existing in the experiments.

In the Eytelwein formula, the depth between the piers is considered the same as that in the normal channel below the piers. This neglects the recovery of any part of the velocity head, and gives a slightly larger contracted area and a smaller velocity than actually exists. It is not practicable to design piers with the proper curves to recover all the velocity head, and the error in neglecting the recovered velocity head in calculating the velocity between the piers is not appreciable. The coefficient of contraction in the Eytelwein formula can easily be estimated by comparison with the coefficient of a submerged orifice with similar characteristics. The two coefficients in the author's formula would have to be assumed for any conditions different from those of the experiments, and there seems to be no criterion available in selecting these coefficients. It is the writer's belief that, though credit is due the author for the thoroughness of his experiments, the formula deduced therefrom, if applied to other conditions, may lead to entirely wrong conclusions.

Mr. Goodrich. R. D. GOODRICH,* M. AM. SOC. C. E. (by letter).—The writer was engaged in a special study of the effects of an encroachment of a proposed building on a river, at a bridge on the main street of a certain city, when the *Proceedings,* containing Mr. Nagler's paper with its discussion of existing bridge pier formulas was issued. Consequently, the results of his experiments and his new formula were most opportune, and were studied with the greatest interest. After some consideration of the theory used in the development of the formula, and some comparison of the results obtained by the various formulas given, the writer decided to use it in the preliminary computation of the back-water which might be expected from the proposed encroachment.

The discharge of the river at the highest stage known was taken from United States Geological Survey reports, and a probable mean velocity of 4 ft. per sec. was determined. The back-water caused by the bridge, with its one center pier, was then determined, the channel having a clear width of about 206 ft. The back-water, with the width

* Lansing, Mich.

of the channel reduced to 170 ft. was then computed, and the difference of 3 in. in the two cases was used as the height of the back-water to be expected from the construction of the building. By assuming the velocity of approach to be the same in both cases, this factor can be eliminated before substituting numerical values, and the increase in back-water, due to an encroachment at one side of the channel, can be found directly. Mr. Goodrich.

Having completed these preliminary calculations, the city attorneys drew up their bill of complaint, and a temporary restraining order was granted to prevent further construction. Among other grounds for injunction, this additional 3 in. of back-water was set up as one cause of complaint. The work of preparing the case for an early hearing was then taken up in earnest, but a great disappointment was encountered.

Surveys were made of the river immediately above and below the bridge in question, new data were collected, and all the computations were reviewed. As the revision progressed, it was found that the velocity in the channel above the bridge would be more than 5 ft. per sec. With this velocity, however, the formula gave a negative back-water. The computations were reviewed again and again, but the formula failed entirely just outside the actual range of Mr. Nagler's experiments.

At this writing (July, 1917), the final hearing is being had, and the testimony of a well-known and experienced hydraulic engineer, employed by the city, shows that about 1 in. of additional back-water may be expected to result from the erection of the building. The explanation to the Court of the disappearance of the other 2 in. of back-water is not anticipated with any great pleasure, but it will be easier than to tell how the water is piled up higher below the bridge than above it. The lesson on the use of untried formulas is so obvious that it is passed on for whatever it may be worth. It is fortunate for the writer that the character and limitations of the author's new formula were discovered previous to the hearing, and a much more embarrassing situation thus avoided.

A full publication of Mr. Nagler's experiments would have been of the greatest value in studying this problem, and it is to be regretted that such a paper as this is not published with the tables in full for the benefit of the majority of the members of the Society who reside outside of New York City and cannot avail themselves of the privileges of the Society Library. The writer is well aware of the great care with which the author conducted his experiments, and it is to be hoped that, with further work along the same line, a more logical formula will be evolved in time.

Mr. Goodrich. The following is a statement of the conditions in the special problem which led to this discussion:

Discharge, $Q = 19\,500$ sec-ft.;
Net width at bridge, $W = 203$ ft.;
Mean depth at bridge, $H_2 = 16.4$ ft.;
$C = 0.894$ (using for the pier the D nose and the A tail as described in the paper);
Area of channel above bridge $= 3\,680$ sq. ft.;

$$V = \frac{19\,500}{3\,680} = 5.3 \text{ ft. per sec.};$$

$$h = \frac{V^2}{2\,g} = 0.4367,\ 1.8\,h = 0.786,\ 0.3\,h = 0.131.$$

Assume the velocity of retreat to be equal to the velocity of approach; then, from the formula for Q, on page 360,

$$H = \frac{Q^2}{2\,g\,C^2\,W^2\,(H_2 - 0.3\,h)^2} - 1.8\,h$$

$$= \frac{19\,500^2}{64.4 \times 0.894^2 \times 203^2\,(16.4 - 0.13)^2} - 0.786$$

$$= 0.686 - 0.786 = -0.100.$$

Mr. Lane. E. W. Lane,* Jun. Am. Soc. C. E. (by letter).—The writer has studied this paper with a great deal of interest, as he has been carrying on a series of experiments along a similar line for several years. The data presented are of great value, as they constitute practically the only reliable information on the subject which has been published, and Mr. Nagler is to be commended for the service which he has rendered in making it available to engineers. Some of the conclusions are somewhat unexpected, particularly those regarding the efficiency of the semicircular ends (Type B), the 90° point up stream (Type C), and the effect of the shape of the down-stream end of the pier. In general, the writer agrees with the statements in the paper, but on several important points his observations and experiments have led to conclusions differing somewhat from those of the author.

Experiments on models, such as those described in this paper, are valuable, as they often point out the solution of a problem when an investigation on full-sized apparatus is impossible. Because of the many factors involved in most hydraulic phenomena, however, it is unsafe to apply the results of such experiments directly to actual conditions. Reliable conclusions can only be reached by making a careful analysis of the results of the experiments, to arrive at the underlying causes, and, from a knowledge of these, determining

* Lafayette, Ind.

how far these results may be modified by the larger scale of the actual case. Mr. Lane.

Although the title of the paper is "Obstruction of Bridge Piers to the Flow of Water", and the formula derived by the author gives the height of the back-water caused by them, the writer believes that the increased height of the water surface above a bridge may be influenced by other factors than the size and shape of the piers. The principal causes of back-water are:

(1).—The reduction of the cross-sectional area of the stream as it flows under the bridge;

(2).—The effect of the nose of the pier;

(3).—The effect of the tail of the pier;

(4).—Friction, both of the water against the bottom and sides of the channel and internal frictions in the water itself.

The reduction of the cross-sectional area of the stream as it flows under the bridge is usually the most important factor causing back-water. In this reduction, the piers often play a minor part, their effect sometimes being absent altogether. Perhaps the most frequent cause of serious back-water is the construction of highways or railroads across the bottom land on high fills, compelling the water, which, in times of flood, formerly flowed over the entire valley, to pass through the comparatively narrow opening at the bridge. This was the condition in the Lackawanna *vs.* Erie case cited by the author, and in several other instances which have come to the writer's attention.*

When the cross-section of a stream is thus reduced, in order to flow through the smaller area, the velocity of the water must be increased, and a backing up above the opening occurs until the water reaches an elevation sufficient to create this higher velocity. The change from elevation head to velocity head takes place according to the well-known theorem of Bernouilli.

The action of the nose of the pier is to cause the water to flow through an area smaller than the actual area between the piers. One who has closely observed the action of a bridge pier in times of high water has no doubt noticed that a considerable space adjoining the pier is filled with eddies, in which the motion of the water has only a small component in the direction of the flow of the stream. This effect is particularly noticeable where the nose of the pier is poorly designed, or where the piers are not properly set with reference to the direction of the current. In many cases the abutments have a similar action, particularly if they project considerably into the stream channel. Because of the presence of this eddy-filled

* Report of the Chief Engineer of the Miami Conservancy District, 1916, Vol. I, pp. 67–71. A Report to the Mayor and City Council on Flood Prevention for the City of Columbus, Ohio, Alvord and Burdick, September 15th, 1913, pp. 80–89.

Mr. Lane. space, the area through which the flowing water must pass is less than the actual cross-section of the water under the bridge, and the velocity of the water, therefore, must be correspondingly greater, necessitating a greater heading up above the bridge. As the height varies as the square of the velocity of the water, it is evident that the effect of the nose of the pier may be an important factor.

A striking confirmation of this explanation of the effect of the nose of the pier is given in Fig. 8, showing the stream lines about the model piers. In all cases, with two exceptions, for piers having tails of the same type, the coefficients grow less as the area shown by the sketch to be occupied by eddies increases. For example, of the piers having tails of the *B* type, Fig. 8 shows that when arranged in decreasing order of area occupied by eddies, they are as follows: *AB, CB, DB, BB,* or *FB*. The lines about *AB* are not shown, but they may be judged from those about *AA*. This order is the same as when these piers are arranged in increasing order of coefficients. The only exceptions to this rule are Models *EA* and *FA*, which seem to be the same as the "inconsistencies" mentioned by the author.

The third factor, the effect of the tail of the pier, is more difficult to analyze. Perhaps the most common explanation would be that the higher coefficients of a certain pier form are due to the recovery of head resulting from the gradual enlargement of the cross-section of the stream. Although the effect may be due in some measure to this cause, the writer believes that the action of the tail of the pier is similar to that of the nose, namely, its influence on the effective cross-section of the stream. Below a bridge pier, in times of high water, eddies or vortices are formed by the action of the flowing water on the comparatively still water in the space directly behind the piers. The vortices on the two sides of the pier diverge as they pass down stream, in extreme cases meeting those from adjacent piers some distance below. The action of these eddies further reduces the effective area of the bridge opening and increases the back-water. This seems to be the author's conclusion from the statement on page 337: "The pier with a tail which follows the diverging stream lines most closely, reducing the volume of eddying water to a minimum, should be most efficient at the down-stream end." Diagrams made during the flood of the Seine in 1910, showing the formation of these eddies by the bridges of Paris, have been published.*

That the effect of the tail of the pier is of much less importance than that of the nose is shown by a comparison of the coefficients in Fig. 20. The difference between the coefficients of the most and least efficient piers of any set having tails of the same type, ranges

* *Annales des Ponts et Chaussées*, July–August, 1911, Ninth Ser., Vol. IV, Part IV, pp. 11–53.

from 0.061 to 0.079, with an average of 0.070; and for the same shaped nose and different tail shapes, it varies from 0.016 to 0.040, or, neglecting the apparently inconsistent results for Piers *EA* and *FA*, it ranges from 0.016 to 0.023, with an average of 0.020. The relative importance of the nose and tail of the pier, therefore, is about as 7 to 2. Mr. Lane.

The effect of friction loss has been little discussed by the author, because, in his experiments, it was negligible. In actual cases of back-water, however, this may become an important factor. The piers of bridges are often protected from undermining by large piles of loose stone around the base, or the bottom of the stream may be rough or filled with piling. In connection with the flood prevention work of the Miami Conservancy District,* the writer had occasion to measure the flow through the contraction of a river channel caused by the construction of a bridge. As there were no piers in the stream, the entire change of elevation head, not accounted for by the increased velocity, was lost in friction. The high velocity of the water had washed away the finer material, leaving the bottom covered with rocks, averaging, perhaps, 9 in. in diameter. The only condition tending toward high friction loss in this measurement, which would probably not be present in most cases of back-water, was the comparatively small depth of the flowing stream, which varied from 4 ft. above the bridge, to 2 ft. in the most contracted section. Of the total drop of 1.36 ft., 0.43 ft. was changed into velocity and the remaining 0.93 ft. was lost in friction. It is evident, therefore, that in some cases friction may be of considerable importance.

If the foregoing analysis correctly represents the effect of the factors causing back-water at bridges, the rational formula for this phenomena would be developed as follows: Suppose that the first of the four causes is the only one acting, and that there is no recovery of head below the piers. The formula, using the author's notation, would be, by Bernouilli's theorem:

$$h + y + \frac{v_0^2}{2\,g} = h + \frac{v_2^2}{2\,g},$$

or,

$$y = \frac{v_2^2 - v_0^2}{2\,g} = \frac{V_0^2}{2\,g}\left[\left(\frac{W\,(h + y)}{w\,h}\right)^2 - 1\right].$$

If we consider also the effect of the nose and tail of the pier, as described above, the effective area between the piers becomes $m\,w\,h$,

* A complete description of this measurement will be published in the Technical Report of the Miami Conservancy District, Part IV, "Calculation of Flow in Open Channels."

Mr. Lane. where m is the ratio of the effective to the actual width between the piers, and

$$y = \frac{v_0^2}{2\,g}\left[\left(\frac{W\,(h+y)}{m\,w\,h}\right)^2 - 1\right]$$

which, expressed in the terms of Q, is

$$y = \frac{Q^2}{2\,g}\left[\frac{1}{(m\,w\,h)^2} - \frac{1}{W\,(h+y)^2}\right].$$

This is the form ascribed to Debauve, although the writer, as will be shown later, believes that it was first derived by d'Aubuisson. The writer does not agree that this is the same as the formula accredited to Eytelwein, as it gives results $\left(\frac{h+y}{h}\right)^2$ times as large. In actual cases, the channel is not rectangular, as is assumed in this notation, and a more practical formula results by replacing $W\,(h+y)$ by A_0, the cross-sectional area of the stream above the bridge, and $w\,h$ by A_2, the area between the piers, making the foregoing formula

$$y = \frac{Q^2}{2\,g}\left(\frac{1}{m^2\,A_2^{\,2}} - \frac{1}{A_0^{\,2}}\right).$$

In this formula, as the coefficient, m, is the ratio of the effective area to the actual area under the piers, other things being equal, the opening having the greatest number of piers would have the greatest part of the area filled with eddies, and, therefore, the lowest coefficient. In the same way, within certain limits, the wider the bridge pier, the wider would be the eddy-filled area about it, and the lower would be the coefficient. As the author's formula, as will be shown later, differs only slightly from that just given, the writer does not agree with his conclusion that the size and spacing of the piers would have no effect on the coefficient of discharge of bridge openings. The experimental evidence offered does not necessarily lead to that conclusion. Although the coefficient of a small orifice, or even one 4 ft. square, may be about 0.62, one would hardly expect that, with the same head, the width at the *vena contracta* of a stream, issuing from an orifice 100 ft. wide, would be only 62 ft. The action would probably be more like that of a weir with end contractions, where the effect of the contraction at each end reduces the effective length by 0.1 H. In the case of the trash racks cited by the author, to give the same "total area of obstruction", with twice the number of bars, requires that the width of each bar be just half as great; and the loss occasioned by having twice the number of bars, may be offset by their being only half as wide.

The modification of the formula developed previously to take account of friction must necessarily be largely a matter of judgment, as

so little is known of its influence that exact analysis is impossible. An interesting method of introducing this factor was developed by Ivan E. Houk, Assoc. M. Am. Soc. C. E., in connection with the studies of the Dayton flood.* This method was used in determining the discharge of the Miami River and its tributaries from measurements of the drop at bridge openings, but is not so readily applied to the reverse operation of finding the height of back-water resulting from a given discharge. Perhaps the most practical way of taking account of friction would be to decrease the coefficient, *m*, by an amount dictated by judgment, based on the conditions existing at the particular opening under consideration. Mr. Lane.

The formula developed previously may also be expressed in the form,

$$Q = C\,A_2\sqrt{2\,g\left(H + \frac{v_0^2}{2\,g}\right)},$$

where H is the drop and corresponds to the y of the preceding formula, and the symbol, C, is used to express the contraction coefficient in place of m. It will be seen, therefore, that the discovery that the "discharge might more correctly be represented as that through an orifice of a sectional area equal to the minimum stream width multiplied by the minimum stream depth, under a head, H, equal to the back-water caused by the bridge pier (corrected for velocity of approach)", is not new, having been known not only by Unwin and Frizell, but was predicted by d'Aubuisson (or Debauve), was published by d'Aubuisson in his "Treatise on Hydraulics" in 1840, and is a direct application of the Theorem of Bernouilli, which was proposed nearly 200 years ago.

The writer agrees that this proposition is the rational basis for the back-water formula, and that the formulas of Weisbach and Merriman, when applied to bridge openings, violate "a very fundamental principle of hydraulics." In practical cases, however, that part of the discharge which, according to these formulas, is assumed to flow over a weir, is usually relatively small, and does not introduce great error in the result. It is interesting to note that, in the tenth edition of his "Treatise on Hydraulics," Professor Merriman has adopted the form ascribed by the author to Eytelwein, which the author believes, "in all probability, is the most correct formula which has been derived, taking the preceding theory as a basis." The writer believes, as will be shown later, that this formula is also erroneous. Experiments undertaken by him, which have not yet been completed, indicate that, for certain conditions of flow through contracted openings, the Weisbach formula is not only empirically, but also theoreti-

* Report of the Chief Engineer of the Miami Conservancy District, 1916, Vol. I, pp. 67–71. The Official Plan of the Miami Conservancy District, Vol. I, pp. 67–71.

Mr. Lane. cally correct. These conditions, however, do not occur in the ordinary case of back-water at bridges.

The formula developed previously differs very little from the author's final form, as it can be expressed by the following equation:

$$Q = C\, W \sqrt{2\,g}\left(H - \alpha \frac{v_2^{\,2}}{2\,g}\right)\sqrt{H + \beta \frac{v_1^{\,2}}{2\,g}}$$

where α and β are zero and 1.0, in place of the 0.3 and 1.8, respectively, adopted by the author. The justification for the introduction of the term, $\alpha \frac{v_2^{\,2}}{2\,g}$, seems to be (from Fig. 19) that there is a rise in the water surface down stream from the piers. That this occurred in the experimental flume, the writer has no doubt, but he does not believe that it is representative of the usual conditions of back-water. It should be noticed that the waterway on each side of the model pier had a width of only about 1.6 times the width of the pier. The wave formations about the pier, therefore, would occupy a much greater relative space in this flume than in actual conditions, where the corresponding space would probably be about six to ten times the width of the pier, and a rise, which in actual conditions might take place only in the immediate vicinity of the pier, and not affect appreciably the greater part of the flow, in this narrow flume would occupy the whole width.

The only information bearing on the question of the rise occurring below bridge openings, of which the writer has knowledge, was that collected under the direction of Mr. Ivan E. Houk, in connection with the investigation of the Dayton flood by the Morgan Engineering Company. Detailed surveys were made at a number of bridge openings, and in only one case was there any evidence that the elevation of the water surface a short distance down stream from the opening was higher than the lowest point of the water surface in the opening. Practically, however, the term, $\alpha \frac{v_2^{\,2}}{2\,g}$, is usually a very small correction, for, even with v_2 equal to 16 ft. per sec., this correction would be only 1.2 ft., and, as it is subtracted from H_2, which is usually comparatively large, it has relatively little effect. The effect of the $1.8 \frac{v_1^{\,2}}{2\,g}$ is greater, as it is added to the observed head, which is never more than a few feet. On the other hand, the discharge is a function of $\sqrt{H + 1.8 \frac{v_1^{\,2}}{2\,g}}$, and the importance of the velocity of approach correction, therefore, is reduced.

From his search of the literature dealing with the flow of water through contractions in a channel, the writer has not arrived at quite the same results as the author regarding the origin of the formulas quoted. Perhaps the author had access to information not available to the writer, but, inasmuch as certain statements already printed in the publications of this Society differ somewhat from those in this paper, it may be well to have the origin of the various formulas established. The late William R. Hutton, M. Am. Soc. C. E., in his paper "On the Determination of the Flood Discharge of Rivers and of the Back-water Caused by Contractions,"* writes as follows: Mr. Lane.

"To determine the increased rise which would be caused by the contraction of the water-way by the works of the Lackawanna Company, two formulas are used, which furnish very different results. The first is that of Eytelwein, the second is quoted from Debauve (*Manuel de l'Ingénieur des Ponts et Chaussées. Ponts et Maçonnerie*), but had long before been given by d'Aubuisson. The great discrepancy in the results given by these two formulas (the rise y by the Eytelwein formula would be 2.3 ft., by that of d'Aubuisson 4.3 ft.), both of them founded upon the same general principle, has suggested an examination of the process by which they were constructed, which seems to be worth recording.

"Dubuat, with whom the method originated, assumes that there will be no considerable change of velocity in the section above the contraction. Consequently the velocity in the contraction will be to that above it, inversely as the sections, $v_{11} = \frac{v_0 W}{w}$, and the difference of the heights to which these velocities are due will be

$$y = \frac{v_{11}^2 - v_0^2}{2\,g} = \frac{v_0^2 W^2}{2\,g\,w^2} - \frac{v_0^2}{2\,g} = \frac{v_0^2}{2\,g}\left(\frac{W^2}{w^2} - 1\right);$$

and the coefficient which is to compensate for the contraction of the stream in the narrow water-way is applied to the resulting height, so that in its final form we have $y = \frac{v_0^2}{m^2\,2\,g}\left(\frac{W^2}{w^2} - 1\right)$.

"In the first edition (1801) of his *Handbuch der Mechanik und der Hydraulik,* Eytelwein follows Dubuat in neglecting the diminution of the velocity above the contraction, caused by the raising of the surface, but he applies the correction for contraction to the narrow section only. Thus making

$$y = \frac{1}{m^2}\left(\frac{v_0\,W}{w}\right)^2 - \frac{v_0^2}{2\,g} \text{ or } \frac{v_0^2}{m^2}\left(\frac{W^2}{w^2} - \frac{1}{2\,g}\right);$$

m being in this case $\sqrt{2\,g} \times 0.95$.

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* *Transactions*, Am. Soc. C. E., Vol. XI, p. 239.

Mr. Lane.

"In the edition of 1843 of the same work he makes

$$v_{11} = \frac{v_0 Q}{w(h+y)} = \frac{v_0 W h}{w(h+y)} \text{ and } y = \frac{v_0^2}{m^2}\left(\frac{W h}{w(h+y)}\right)^2 - \frac{v_0^2}{m^2}$$

or

$$\frac{v_0^2}{m^2}\left(\frac{W^2 h^2}{w^2 (h+y)^2} - 1\right),$$

as quoted by General Gilmore.

"It will be observed that v_0 being the original mean velocity, v_{11} (that between the piers) is not $\frac{Q}{w(h+y)}$ but simply $\frac{Q}{w h} = \frac{v_0 W h}{w h} = \frac{v_0 W}{w}$. Again, as the second member of the primary equation is the head due to the velocity above the contraction, it is equal to $v^2 \left(\frac{W h}{W(h+y)}\right)^2 = v_0 \frac{h^2}{(h+y)^2}$, which does not vary with the coefficient of contraction. Substituting we have $y = \frac{v_0^2}{2g}\left(\frac{W^2}{m^2 w^2} - \frac{h^2}{(h+y)^2}\right)$, m being the usual coefficient of contraction applicable to the character of the entrance to the contraction. This form involves smaller numbers than the formula quoted by General Gilmore from Debauve, but it is substantially the same."

From this quotation it seems that the formula ascribed to d'Aubuisson, according to Mr. Hutton, was proposed by Dubuat, and that ascribed to Eytelwein is not the same as either of those quoted by Mr. Hutton from Eytelwein's "Handbuch der Mechanik und der Hydraulik", but is a modification of them which was proposed by Mr. Hutton, and which the writer believes is based on erroneous assumptions.

The following is quoted from "A Treatise on Hydraulics," by J. F. d'Aubuisson, published in 1840, and translated by Joseph Bennett in 1852, "Back-water produced by contracting the water-way," page 188:

"The height of this fall will also be given by the equation

$$p' = \frac{v^2}{2g} - \frac{v_0^2}{2g} = \frac{Q^2}{2g}\left(\frac{1}{s^2} - \frac{1}{s_0^2}\right).$$

Let x be the height of fall, L the mean breadth of the stream above the contracted space, l the width of the contracted part, and h, the depth of water in that part; its section, s, will be $l\ h$, or rather $m\ l\ h$, m being the coefficient of contraction at the entrance; for the section, s_0, of the current immediately above the fall, we have $L\ (h+x)$, $h+x$ being the depth of water there and L the breadth. Thus, observing that x is the slope designated above by p' we shall have

$$x = \frac{Q^2}{2g}\left(\frac{1}{m^2 l^2 h^2} - \frac{1}{L^2 (h+x)^2}\right)."$$

The writer was not able to determine whether the work of Debauve antedates that of d'Aubuisson, but it is evident that they both used the same form, and that Mr. Hutton believed that d'Aubuisson was the originator of it. Mr. Lane.

A close examination of the reasoning of Mr. Hutton, in which he develops the formula,

$$y = \frac{v_0^2}{2\,g}\left(\frac{W^2}{m^2\,w^2} - \frac{h^2}{(h+y)^2}\right)$$

will show its incorrectness, from a theoretical standpoint. According to Mr. Hutton, "It will be observed that v_0 being the original mean velocity, v_2 (that between the piers) is not $\frac{Q}{w\,(h+y)}$ but simply

$$\frac{Q}{w\,h} = \frac{v_0\,W\,h}{w\,H} = \frac{v_0\,W}{w}."$$

The equalities given are erroneous, as $\frac{Q}{w\,h}$ is not equal to $\frac{v_0\,W\,h}{w\,h}$ but rather to $\frac{v_0\,W\,(h+y)}{w\,h}$, which is $\frac{h+y}{h}$ times as large as $\frac{v_0\,W\,h}{w\,h}$, and, therefore, accounts for the formula derived by Mr. Hutton and ascribed to Eytelwein, giving results $\frac{h^2}{(h+y)^2}$ times as large as that of Debauve (or d'Aubuisson), as previously mentioned.

In conclusion, though the writer does not believe that it has been demonstrated that all the existing formulas are erroneous, nor that the author's formula is "far more adequate and simple than any of the existing formulas", as it differs from one of the existing forms only by the addition of the empirical corrections, $0.3\,\frac{v_2^2}{2\,g}$ and $8.1\,\frac{v_1^2}{2\,g}$, the author has undoubtedly pointed out the true action of the water in flowing through a bridge opening, and it is to be hoped that the formulas based on the combined weir and orifice theory, now so commonly used, will be rapidly discarded. It is doubtful if the results obtained from such small-scale models justify the adoption of the corrections mentioned above, when applied to actual cases of back-water, and the formula of Debauve (or d'Aubuisson), when expressed in terms of the area of the stream above and between the piers, is much more simple and practical. The effect of these corrections in most actual cases of back-water, however, will be so slight that the author's coefficients probably could be used with either formula, within the limit of accuracy which it is possible to obtain in practice. These coefficients supply a long felt need, as they offer a convenient index to the efficiency of the various types of piers, which will be of great value to designers of bridges. Although

Mr. Lane. the experiments cover most practical forms of piers, a number of points, such as the effect of the size and spacing of the piers, were not determined, and it is hoped that these will be investigated in the near future.

Mr. Merriman. MANSFIELD MERRIMAN,* M. AM. SOC. C. E. (by letter).—The conclusions derived by the author regarding the relative efficiencies of piers of different forms are interesting and valuable. Although most of them have heretofore been known in a general way, this seems to be the only series of experiments which furnishes numerical comparisons, and hence the author has rendered a distinct service to hydraulic and bridge engineers.

Fig. 19 is a good representation of the profile of the stream at the side of a pier; here, however, the hump at the nose of the pier and the depression along the side are local phenomena which are not seen at a distance of 10 or 15 ft. from the side of a common bridge pier. For two piers, 100 ft. or more apart, the profile of the stream surface half way between them is observed to be a straight line, and numerous other profiles do not resemble Fig. 19. Fig. 13, which the author criticizes, was not intended to represent a profile of the stream at the side of a pier, as the pier lines are drawn broken; it was intended, however, to represent approximately an average of all the profiles between two piers. It appears then, as the author admits, that there is some justification for the formula on page 352, although, like many hydraulic formulas, it cannot be regarded as theoretically perfect. That formula, however, is an awkward one from which to compute values of the back-water rise, H, and, therefore, it is to be desired that a more satisfactory one may be established.

The writer is inclined to the opinion that the formulas given on page 359 contain defects which do not justify their universal application, but he is very glad that the author has deduced such an extended series of coefficients for them. The main use of these formulas is, of course, for the computation of values of the back-water rise, H, from given values of Q, W, and H_2. Had the author given a numerical example showing how this may be done, it would have added to the interest of his valuable paper.

Mr. Frankland. F. H. FRANKLAND,† ASSOC. M. AM. SOC. C. E.‡—The author states that there are very few experimental data on the subject of the obstruction of bridge piers to the flow of water.

As evidence which would seem to indicate that the mathematical solutions of the problem and existing formulas are unreliable, attention is called to the writings of M. Bossut, the well-known French engineer, wherein his mathematical analysis of the problem led him to conclude that the pier nose should be a right-angled triangle in

* New York City.

† Kansas City, Mo.

‡ Now M. Am. Soc. C. E.

section. On the other hand, M. Dubuat, in his "Principles of Hydraulics," gives a mathematical solution which suggests a section having convex curves for the pier-nose faces. Mr. Frankland.

Cresy's well-known experiments on the shape of piers were made many years ago, with various small models, about 15 cm. thick, and placed in a canal between vertical stop-boards which controlled the depth of water. Cresy found that the elliptical section gave better results than any other form, inasmuch as the resistance to flow was less and the stream contraction was a minimum. The worst results, as indicated by these experiments, were shown by a model with concave-faced pier noses.*

Further, to illustrate the fact that previously existing formulas for pier design, in section, are misleading, the speaker has been told by J. A. L. Waddell, M. Am. Soc. C. E., that some years ago, during a controversy before the Secretary of War regarding the then existing Kansas City Southern Railroad Company's bridge at Ohio Avenue, over the Kaw River, at Kansas City, a certain German engineer testified that his mathematical investigations showed that these bridge piers would offer such obstruction to the stream flow as to cause a difference in water level between the up-stream and down-stream sides of the bridge, at high-water stage, of 11 in. Dr. Waddell stated positively that the obstruction would be negligible. Careful investigations made later by the Railroad Company confirmed this statement. In fact, the difference in elevation between the water on the up-stream and down-stream sides of the bridge is barely measurable.

CHARLES EVAN FOWLER,† M. AM. SOC. C. E.—The subject of the proper form of bridge piers has been discussed ever since modern bridge engineering has been practiced. The speaker has covered the practical part of this subject quite fully in his book, "Sub-Aqueous Foundations", in which the essential features are shown by Figs. 324 and 325. Mr. Fowler.

The construction of piers in a stream modifies the usual currents and flow of the water to such an extent as to endanger very often the foundations themselves. Early French writers thought to have solved completely the problem as to the form of a pier to cause the least disturbance, and M. Bossut, from a mathematical investigation, concluded that the starling should be a triangle, the nose being a right angle. M. Dubuat gave another and more nearly correct solution in his "Principles of Hydraulics", the faces of the starling to be convex curves tangent to the faces of the pier.

Experiments recorded in Cresy's "History of Civil Engineering" are partly illustrated in Fig. 21. The models were 15 cm. in

* A résumé of these experiments is given in "Sub-Aqueous Foundations," by Charles Evan Fowler, M. Am. Soc. C. E.

† New York City.

Mr. Fowler. thickness and the velocity of the current was 3.09 m. per sec. The illustrations show the direction and height of the induced currents and eddies, and the relative value of the various forms of the pier cross-sections, the best practical form being the section Fig. 21 (*e*), where the starling is formed of two convex curves tangent to the sides and described on an equilateral triangle. The very best form, however, is the elliptical, Fig. 21 (*f*).

The piers of the Knoxville steel arched cantilever were of the cross-section shown in Fig. 21 (*e*), and were equally spaced by the speaker, thus making the structure symmetrical, and contributing to its architectural appearance.

The late George S. Morison, Past-President, Am. Soc. C. E., also adopted this form for most of the piers built by him in the Missouri and Mississippi Rivers, but, from a point just above high water, he used semicircular ends. This was a very good design, both for practical and esthetic purposes.

Gustav Lindenthal, M. Am. Soc. C. E., stated to the speaker recently, that, in his opinion, long spans would only be possible during the next 20 to 30 years, owing to a future normal advance in the price of steel, due to the very rapid depletion of the raw materials for steel manufacture.

Although the form of piers is vital with long spans, as regards mainly the integrity of the piers themselves, the subject will become of increasing importance as engineers are forced more and more to build short-span structures of reinforced concrete. This type is already being constructed in numerous locations where a thorough investigation of the best form of pier is necessary.

The speaker, however, does not believe that the results obtained from the experiments under discussion are of more than tentative value, owing to the conditions of the tests being wholly unlike those encountered in actual practice, where the materials and methods of pier construction become vital factors of the problem. Then, there are the portions of the cribs below water, rip-rap protection around piers, the character of the river bottom, the scouring actions induced, and many other items, to which consideration must be given. Therefore, it would seem that, until we have the results from a series of experiments, conducted on lines more nearly in accordance with actual conditions, with several piers and on a larger scale, we can only accept those general conclusions which are in reality confirmatory of the findings of the data from Cresy.

The form then, as shown in Fig. 21 (*e*), as adopted by Mr. Morison, and as used by the speaker at Knoxville and elsewhere, would seem to be the logical one to adopt ordinarily, using care at the same time to avoid unnecessary obstruction from cribs, rip-rap, or other protection work.

Mr. Fowler.

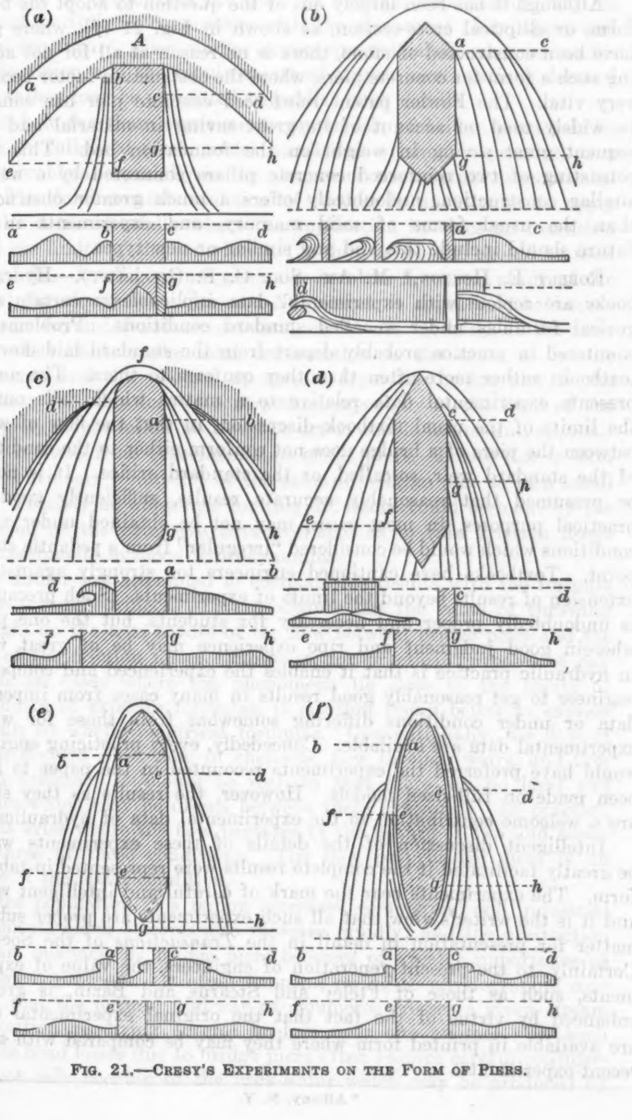

FIG. 21.—CRESY'S EXPERIMENTS ON THE FORM OF PIERS.

Mr. Fowler. Although it has been largely out of the question to adopt the better form, or elliptical cross-section, as shown in Fig. 21 (*f*), where piers have been constructed of stone, there is no reason at all for not adopting such a form for concrete piers, where the obstruction factor becomes very vital. The Fowler patent reinforced concrete pier has come to be widely used on account of its great saving in material and consequent great saving in weight on the foundation bed. This pier, consisting of two reinforced concrete pillars connected by a web of similar construction, undoubtedly offers a much greater obstruction than the usual forms of solid masonry, and experiments in the future should include this and any similar or new types.

Mr. Horton. ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—Hydraulic books are replete with experimental data applicable to certain categorical formulas under so-called standard conditions. Problems encountered in practice probably depart from the standard laid down in textbooks rather more often than they conform to them. The author presents experimental data relative to a matter which falls outside the limits of the usual textbook discussion, in that the flow of water between the piers of a bridge does not conform either to the conditions of the standard weir, so-called, or the standard orifice. It is not to be presumed that reasonably accurate results, sufficiently good for practical purposes, in most cases, may not be obtained under many conditions which would be considered "irregular" from a pedantic standpoint. Textbooks have cautioned engineers too strongly against the extension of results beyond the limits of experiments. Such precaution is undoubtedly proper and necessary for students, but the one point wherein good judgment and ripe experience may be of great value in hydraulic practice is that it enables the experienced and competent engineer to get reasonably good results in many cases from imperfect data or under conditions differing somewhat from those for which experimental data are available. Concededly, every practicing engineer would have preferred the experiments recounted in the paper to have been made on full-sized models. However, the results as they stand are a welcome contribution to the experimental data of hydraulics.

Intelligent discussion of the details of these experiments would be greatly facilitated if the complete results were represented in tabular form. The experiments bear the mark of careful and intelligent work, and it is the writer's view that all such experiments are proper subject matter for presentation in detail in the *Transactions* of the Society. Certainly, to the present generation of engineers, the value of experiments, such as those of Fteley and Stearns and Bazin, is greatly enhanced by virtue of the fact that the original experimental data are available in printed form where they may be compared with more recent experiments.

* Albany, N. Y.

Mr. Horton.

It is to be hoped that the author's original experiments may be presented in full, in connection with the closure of this paper; and that, in the future, when similar papers are published, wherein intelligent discussion and the requirements of future investigations demand it, the author may be permitted to publish such details.

Just how far these experiments on models of bridge piers can be applied to full-sized piers, under conditions arising in practice, is a difficult problem, especially in the absence of the original experimental data. The fundamental condition of success in an attempt to apply hydraulic data, especially where the application involves an extension beyond the limits of the experiments, is that the results should be used intelligently. Intelligent use generally involves a critical study of the effect of the changed conditions. At present, it appears to be impossible to go beyond the statement that, in the case at hand, if the ratio of the obstructed to the total cross-section is the same as in the experiments and the velocity up stream from the bridge pier is the same, then the eddy loss or contraction, especially at the head of the bridge pier, should follow laws and be determinable in amount, approximately, at least, from experiments on model piers. Losses due to skin friction obviously would not be the same in a large pier as in a small model. Possibly an allowance for this difference could be made by the application of the extensive data available as to skin friction losses from experiments in naval hydrodynamics.

The author has attempted to correlate his results with the formula for flow through orifices. This procedure has the value of throwing some light on the true nature of the flow between bridge piers and the nature of the friction losses. However, for practical purposes of calculation, in cases where the phenomena lie close to the limits of applicability of existing formulas, it is often found better to express the result by wholly empirical formulas. Conventionally, the flow of water is divided into: flow in channels, flow through weirs, and flow through orifices. It should be realized that there are no hard and fast lines which may be drawn between these categories. In practice, problems arise where the flow partakes of the nature of two and sometimes three of these conditions. The flow past bridge piers is an example of the latter condition.

As the population in a region increases, commercial and industrial developments along streams also increase rapidly; and the obstruction of stream channels by bridge piers becomes of greater importance as development proceeds. It often happens, owing to physiographic conditions, that the natural highway of commerce crosses a given stream in a position requiring several bridges near one another. In such cases, the head losses due to bridge piers often require careful consideration, not only because of the back-water which may be produced by

Mr. Horton.

improper construction, in flood seasons, and under ordinary flow conditions; but also because of the danger of the formation of jams of logs, débris, or ice, especially where there are several bridges with numerous piers not in alignment. In a recent case which came to the writer's attention, two bridges had been constructed across the outlet of a large lake. The piers were close together, and in such a position that those down stream obstructed the waterway left between those of the up-stream bridge. To the casual observer the head loss appeared to be small. Both measurement and calculation, however, indicated that it was of considerable moment, especially in view of the fact that the back-water which it caused affected the entire perimeter of the lake above, which was about 60 miles in length.

The author states that Gauge No. 3 measures the head up stream from the weir crest at a distance proportional to the ratio of the height of the weir to the height of the experimental weir used by Bazin. It appears to the writer that, inasmuch as the author's weir was lower than that of Bazin, the velocity of approach and surface curvature would be greater for a given depth on the weir, and the gauge should have been placed at the same distance up stream, or else at a greater distance than that of Bazin's.

In Fig. 10, showing the comparison of the experimental friction loss with that obtained by the application of Kutter's formula, the author should state what value was used for the coefficient of roughness, n.

The writer has had occasion to make a study of the comparative results obtained in computing back-water according to the different formulas mentioned by the author. The problem was as follows: A canal discharging 1 500 cu. ft. per sec. had an irregular cross-section with a mean width of 88 ft. and a mean depth of 8 ft. A pier 11 ft. wide was to be erected in the center of the section. It was desired to find the height of back-water or h. The following results were obtained, using the coefficients recommended by the authors of the respective formulas:

Debauve formula	Coefficient = 0.96	$h = 0.107$
Weisbach "	" = 0.95	$h = 0.076$
Neville's "	" = 0.80	$h = 0.160$
Trautwine's table	....	$h = 0.102$
Molesworth's table	....	$h = 0.110$

The tables of Molesworth and Trautwine are based on a formula similar to that quoted by the author on page 350 and attributed to D'Aubuisson. They modify the formula, however, to take into con-

sideration the value of the slope of the stream bed itself. The formula thus assumes this form: Mr. Horton.

$$h = \left(\frac{v^2}{k^2\, 2\, g} + s\right) \left\{ \left(\frac{A}{a}\right)^2 - 1 \right\}$$

Louis D'A. Jackson* also gives a table for computing the value of h, using this same formula, but considering s as having a value of zero.

The Neville formula† is a submerged-weir formula similar to that of Weisbach.

DAVID A. MOLITOR,‡ M. AM. SOC. C. E. (by letter).—The writer had occasion to devote several months to this problem in making computations relative to the Kansas City Northwestern Bridge over the Kaw River at Kansas City, Mo. The case was brought by the Kansas Drainage Board against the Missouri Pacific Railway, and many eminent members of the Profession gave testimony. Mr. Molitor.

The case was extremely complicated, as the bridge in question was located at about 36° with the stream center line, with four piers normal to the bridge axis, and the river on a curve. There were fifteen other bridges up stream from the one in question, all of which influenced the flood stage and back-water conditions involved in the controversy. The last testimony was given in May, 1915, and the State Supreme Court recently decided the case in favor of the Railroad Company.

When Mr. Nagler's paper appeared, in May, 1917, the writer naturally gave it a careful reading, and attempted to incorporate the experimental data into his former calculations, but without success.

Although the author gives a formula for back-water height produced by bridge piers, his experiments certainly did not warrant the broad conclusions which he draws, and it is questionable whether the main purpose of the experiments "to determine the relative obstruction to the flow of water offered by piers of different designs" was answered satisfactorily.

The most valuable contribution to science, of any experimental research, is always a publication of the complete observations, as these constitute the best evidence. An author's deductions may be wrong, but the experiments, if published, may enable the reader to find the trouble. In the present case, only enough data were published so that the thread of the argument could be followed.

In so far as the author's experiments include only one span length of 2.138 ft. obstructed by a single pier 0.5 ft. wide, he has eliminated all the variables in the problem except depth of water and velocity, so that his empiric constant is not put to the severe test of satisfying all the conditions generally imposed. Therefore, the summary dis-

* "Hydraulic Manual", Tables, p. 111.

† "Hydraulic Tables, Coefficients and Formulas," p. 142.

‡ Detroit, Mich.

Mr. Molitor.

missal of all previous formulas wherein the coefficient varies as much as 9%, was entirely unwarranted in the face of his experiments.

The further conclusion that the older formulas "have been disregarded by modern investigators, as the losses at the tail of the pier and the standing wave have been entirely neglected", is also incorrect, when ordinary bridges are under consideration. The writer has observed many bridge piers in swift currents, but has never seen a case where these conditions extend over the span from pier to pier. The heading up above a pier and the standing wave are purely local conditions at a pier, but, in the author's flume, they became general conditions, owing to the small width of the flume and the comparatively large obstruction offered by the pier. This, then, explains why the experiments did not fit any hitherto known formula, and necessitated the introduction of two constants, α and β, to bring about the required agreement; at the same time they rendered the formula worthless for general application.

There also appears to be an error in the algebraic substitution of the value $D = H_2 - \alpha \frac{V_2^2}{2g}$ and $H = H_d + \beta \frac{V_1^2}{2g}$ into the general formula $Q = C\,W\,D\,\sqrt{2\,g\,H}$. The final equation should give the value of H_d, and not of H, as given on page 359.

On page 349, H_d, is defined as "the difference in elevation between the bottom of the flume at Gauges Nos. 1 and 2", and hence the final formula in reality gives the bottom drop instead of the heading up due to the pier.

Regarding the author's conclusions, the writer emphatically objects to the first one, on page 362, as nothing of the kind has been demonstrated in the paper.

The second conclusion is true if the second sentence is omitted, as the author's formula is in no wise comparable with existing formulas.

The third conclusion is correct, but the fourth is extremely doubtful, owing to the fact that the experimental conditions are not representative cases when applied to bridges in general.

The phenomena accompanying the flow under a bridge demonstrate clearly that the back-water is essentially produced by the contraction of the channel, necessitating an increased velocity through the contracted area in order that the quantity of discharge may remain constant at all sections above, below, and at the bridge. This is directly observable, without resorting to small experiments for proof, and is shown by numerous photographs collected by the writer. The local effect at a pier, though influencing the back-water, does not extend over the entire span, but acts in the manner of an end contraction, as for a submerged weir. Hence, the empiric coefficient must be dependent on the number of such end contractions, the depth and velocity of

Mr. Molitor.

flow, and the contracted width of channel. Therefore, such a coefficient must be expected to vary between somewhat wide limits, if all these variable conditions are considered, without discrediting the fundamental hydraulic formula to which it is applied.

This should dispel any hope of ever devising a back-water formula applicable to all bridges, which would embrace a coefficient depending only on the shape of the piers.

Mr. Nagler.

FLOYD A. NAGLER,* JUN. AM. SOC. C. E. (by letter).—The object of this paper was to determine the relative effect of different pier shapes in producing back-water. Some seem to have lost sight of this fact entirely, although it was repeated throughout the paper. The experiments were performed in only one channel, using pier models, each of which obstructed 23.4% of the total channel area. Physical limitations prevented the use of other channel widths, or the use of other ratios of obstruction; therefore the writer gave the following cautions:

"The coefficients * * * are primarily valuable because they indicate the relative amount of obstruction offered by the different shapes of piers * * *." (Page 358.)

"Whether the coefficients for the formula will still have the same value for higher velocities is merely a matter of conjecture * * *." (Page 360.)

"The limitations in the size of the flume permitted tests to be made on only one particular width between piers. Whether these coefficients can be extended to other widths between piers is a matter of conjecture * * *." (Page 360.)

"Experimental data are not at hand, however, which will assure the accuracy of the formula, if applied to conditions differing widely from those in the writer's experimental flume." (Page 362.)

The writer, therefore, has made no guaranty to any reader who blindly applies the coefficients and formulas to conditions differing in their hydraulic proportions from those in the flume described by him. Engineers are too prone to select empirical formulas and coefficients from handbooks and apply them to entirely irrelevant cases, never inquiring as to the natural limitations of the applicability which intelligent use would place on them. Intelligent extension of experimental formulas and coefficients to practical problems is the highest type of engineering, but the blind application of formulas smacks of student days.

The writer's formula, based on what he believes to be the correct theory, contains the empirical constants, α, β, and C, the best values for which were determined from this somewhat limited set of experiments in order to present a rational numerical comparison between the different types of piers. It should be evident to any thinking engineer that the coefficients, particularly α and β, would no doubt assume other values under different channel conditions. As no experi-

* Waco, Tex.

Mr. Nagler.

ments were performed in channels of different widths, the selection of proper values of α and β for other cases is left entirely to the judgment of the engineer. $\beta = 1.8$ is a very high velocity of approach correction, and certainly would be smaller for all channels with a smaller percentage of obstruction offered by the pier than was the case in the writer's flume. It might even assume a value as small as unity for channels in which the bridge pier obstructs only 2 or 3% of the stream. All other bridge pier formulas assume $\beta = 1$ for all conditions. $\beta = 1.8$ is simply the best value of that coefficient for these experiments, or for any other problem where the bridge pier occupies approximately 23.4% of the channel. A simple calculation, using this value of β in the writer's formula, reveals the fact that rational positive results will not necessarily be obtained unless the pier obstructs at least 20% of the channel of approach. Mr. Wiley and Mr. Goodrich seem to have discovered this fact with no little alarm to themselves; but it is as unreasonable to assume that a formula, with one value for the coefficient in correcting for velocity of approach, should fit all varying channel conditions, as it would be to expect the medical profession to produce some concoction which would prove a panacea for all ills.

Only with $\beta = 1$ would it be possible for a bridge pier formula of any of the existing types to be generally applicable; and it is known, from the study of channel contractions caused by weirs, that β is always in excess of unity for weirs of appreciable height. The experiments of Fteley and Stearns show that β assumes a value of 2.05 for channels with almost complete contraction, decreasing to values as low as 1.33 for the lowest weir experimented on, indicating that β would decrease to 1 when there was little or no contraction of the channel. The same variation in the value of β would be expected in an adequate bridge pier formula.

In discussing the proper value of β, it is interesting to note, as Mr. Lane suggests, that the writer's formula is quite similar to those of Eytelwein and Debauve. This is true when $\beta = 1$.

Then $$Q = C\,D\,W\sqrt{2\,g\,(H - h_1)}$$

and $$\frac{Q}{D\,W} = V_2 = C\sqrt{2\,g\,H - V_1^2}$$

thence $$H = \frac{C^2\,V_2^2 - V_1^2}{2\,g}$$

in which
V_1 = velocity of approach,
V_2 = velocity in contracted pier section,
Q = discharge,
D = depth at pier section,
W = width at pier section,
H = back-water,
C = coefficient.

Messrs. Wiley and Goodrich both give numerical examples in which the writer's formula indicated negative back-water, using $\beta = 1.8$. In both these problems the hydraulic conditions differ widely from those in the writer's flume, and the obstruction to the normal channel was far less than 20%, which is the limit of applicability of the writer's formula, using $\beta = 1.8$. It is evident that a smaller value of β should have been used. **Mr. Nagler.**

In Mr. Wiley's example, where only 4.7% of the channel area was obstructed, a value of $\beta = 1.04$ would have been selected by the writer, giving a back-water of 0.07 ft., a value very close to that obtained from the actual levels.

In Mr. Goodrich's case, 9.4% of the channel was obstructed, and consequently a value of β of about 1.25 would have been applicable. This gives back-water amounting to 0.14 ft.

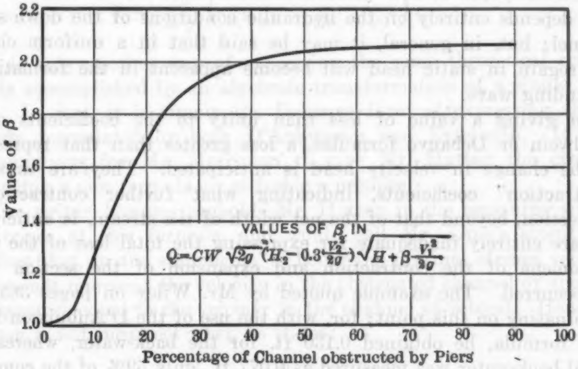

FIG. 22.

In selecting the foregoing values of β, the writer drew a smooth curve (Fig. 22), for different values of channel obstruction, asymptotic to $\beta = 1$ for zero obstruction and $\beta = 2.05$ for 100% obstruction, passing through $\beta = 1.8$ for 23.4% obstruction.

The writer agrees with Mr. Wiley that the value of the coefficient, α, may be found to be other than 0.3 for conditions varying from those in the writer's flume. However, as Mr. Lane observes, the absolute value of this coefficient has but little effect on the results of the computation, especially if the stream depth is appreciable. It is doubtful, therefore, whether speculation as to what other values this coefficient may assume is worthy of comment.

Messrs. Wiley and Lane still favor the use of the Eytelwein and Debauve formulas. If these formulas are generally applicable, they should fit the writer's 256 experiments. They fail to do this. These

Mr. Nagler. formulas are at fault in that they consider hydraulic conditions only as far as the point of minimum section, utterly ignoring the static head regained at the tail of the pier when the section is again enlarged. Thus far the stream has suffered very little energy loss, for energy that does not appear in static head is almost all available in the kinetic energy of the water under the increased velocity as it passes the contracted section. The greater loss, in passing the bridge pier section, occurs (beyond the point of minimum section) in the conversion of the increased kinetic energy back into potential energy. It is true that only a small quantity of this energy can be regained, but this is the point where the efficient tail of the pier plays its part, and the writer has demonstrated that the design of the tail actually does affect the height of the back-water. Whether or not this increase in static head becomes a physical fact (Mr. Lane asserts that it more often does not) depends entirely on the hydraulic conditions of the down-stream channel; but, in general, it may be said that in a uniform channel this regain in static head will become apparent in the formation of a standing wave.

In giving a value of less than unity to the coefficients of the Eytelwein or Debauve formulas, a loss greater than that represented by the change in velocity head is anticipated. They are thus only "contraction" coefficients, indicating what further contraction of the section, beyond that of the net width of the stream, is anticipated, and are entirely inadequate for expressing the total loss of the entire phenomena of the contraction and expansion of the section which has occurred. The example quoted by Mr. Wiley on pages 365-66 is illuminating on this point; for, with the use of the D'Aubuisson-Eytelwein formula, he obtained 0.130 ft. for the back-water, whereas the actual back-water was measured as 0.077 ft., only 59% of the computed value. However, 0.15 ft. loss in static head was measured at the point of minimum section, which value coincides more nearly with the back-water computed by the D'Aubuisson-Eytelwein formula.

In reviewing these older formulas, the writer arrived at different conclusions as to their authorship than those apparently expressed in Mr. Hutton's paper, quoted in detail by Mr. Lane. The discussion by Mr. Hutton not only proved inconsistent with other references on this subject consulted by the writer, but also was inconsistent within itself, in that it contains mathematical errors. It will suffice to point out two of these: The statement, $y = \frac{1}{m^2}\left(\frac{v_0 W}{w}\right)^2 - \frac{v_0^2}{2g}$ (page 375), is correct, according to the premise of Eytelwein, but the equation immediately following it, and offered as a derived equation of the first, is not correct, but should be $y = v_0^2\left(\frac{W^2}{m^2 w^2} - \frac{1}{2g}\right)$ instead of

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$y = \frac{v_0^2}{m^2}\left(\frac{W}{w^2} - \frac{1}{2\,g}\right)$. Hutton says, "Eytelwein * * * applies the correction for contraction to the narrow section only." There appears still another error in the statement, $v_{11} = \frac{v_0\,Q}{w\,(h + y)}$; this should have been $v_{11}{}^2 = \frac{v_0\,Q}{w\,(h + y)}$, for this is the value which is used in the succeeding formula.

Mr. Merriman adopts the formula of A. Debauve,* preferring to call it "Hutton's Formula". Hutton was simply responsible for changing the Debauve formula:

$$y = \frac{Q^2}{2\,g}\left(\frac{1}{m^2\,w^2\,h^2} - \frac{1}{W^2\,(h + y)^2}\right)$$

to

$$y = \frac{v_0^2}{2\,g}\left(\frac{W^2}{m^2\,w^2} - \frac{h^2}{(h + y)^2}\right).$$

This is only another way of expressing the same theoretical relation, and is accomplished by an algebraic transformation of a single operation. In fact, it is simply the Debauve form of expressing theories already formulated by both D'Aubuisson and Eytelwein. It seems presumptuous for Mr. Merriman to give Mr. Hutton the credit of advancing a new formula on so feeble a premise.

The writer, however, upholds Mr. Hutton in the reasoning whereby he arrives at this formula, and would call Mr. Lane's attention to the fact that, in the statement "v_0 being the original stream velocity", v_0 does not represent the velocity in the obstructed channel of approach, but becomes equal to the velocity in the channel of retreat. Hence Mr. Hutton is not incorrect when he states:

$$\frac{Q}{w\,h} = \frac{v_0\,W\,h}{w\,h} = \frac{v_0\,W}{w}.$$

For, as v_0 = the velocity in the channel of retreat, and $W\,h$ = the area of the channel of retreat, Q is then equal to the area of the section multiplied by the velocity in that section $= v_0 W\,h$. $v_0 W\,(h + y)$, which Mr. Lane claims is equal to Q, represents the multiplication of the velocity in the channel of retreat by the area of another section, namely, the channel of approach. This is fundamentally wrong, and the writer is of the opinion that Mr. Hutton is absolutely right.

The writer had hoped that there would be some discussion presenting experimental data as to the effect of the number, size, and spacing of piers offering the same total obstruction. Mr. Lane has stated that he does not agree with the writer in his conclusion that,

* "Treatise on Hydraulics," Tenth Edition.

Mr. Nagler. in the absence of definite experiments, it would not be unreasonable to assume that there would be no change in the value of the coefficients if the number of piers was increased, retaining the same total obstruction to the stream channel as before; yet he cites, in defense of his position, the writer's illustration of the trash racks, where, regardless of the number, spacing, and size of the individual bars, the loss through the racks was always the same, provided there was always the same total area of obstruction. There should be no disagreement on this point, for the latter is exactly the illustrated conclusion of the writer. The problem itself is a complicated one; for, dividing the total loss in passing a bridge pier into loss at the nose and loss at the tail of the pier, the loss at the nose would seem to increase with the number of piers, though that at the tail would seem to vary inversely as the number of piers. The losses at the nose are revealed in the further contraction of the stream channel beyond the net width at the bridge pier section; assuming, therefore, that the piers are of appreciable size, and, regardless of number, offer the same obstruction to a given channel, the number of these contractions varies directly as the number of piers. On the other hand, if we consider the losses at the tail of the pier to be represented by the volume of the triangular prism of eddying water at the down-stream end, this volume, and consequently the losses, varies inversely as the number of piers, being greatest for a single pier, then decreasing as the piers increase in number, and disappearing entirely if we assume an infinite number of piers. This proposition is of easy geometrical proof. These two opposite tendencies in producing loss in a bridge pier section, if the number and width of piers is varied, maintaining the same total net obstruction to the stream, tend to counterbalance each other; whence the coefficient to be applied in solving for the height of back-water might be the same, regardless of the number of piers. It is to be hoped that large-scale experiments will be performed in the near future, which will settle this question. However, the writer does not anticipate, as Mr. Fowler desires, that it will be possible to perform a set of experiments on a large scale duplicating actual conditions encountered in practice. Such a set of experiments would be almost inconceivable, if it were required that all actual bridge pier conditions be duplicated, for bridge pier sections seldom even duplicate themselves. The main value of the set of relative coefficients offered by the writer lies in the fact that they establish a basis on which an intelligent engineer may select a coefficient which will apply to the modifications in the problem at hand. For instance, he may readily estimate how effective a square crib near the channel bottom may be in reducing the coefficient of a round-nose pier; or may arrive at a reasonable coefficient (approximately 0.88) for the Fowler patent reinforced concrete pier.

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The writer's experimental data are filed in the archives of the Society. Mr. Goodrich states that the "well known and experienced hydraulic engineer employed by the city", to whom he refers, had access to these data, and, the writer is informed, used them in arriving at his conclusion that the obstruction in the case cited would cause back-water amounting to about 1 in. The writer cannot conceive how Mr. Goodrich computed the value of 3 in. of back-water in his first set of calculations. The value by the formula, with a velocity of approach correction equal to unity, should give the maximum possible results, and certainly larger than the writer's formula with a coefficient of 1.8, which Mr. Goodrich says he used. The highest thus obtainable is but little more than 2 in., and the writer is of the opinion that the original claim of 3 in. of back-water fails to have the support of any computations by the writer's formula.

The writer notes that although Mr. Merriman states that the profile of Fig. 13, copied from the ninth edition of his "Treatise on Hydraulics", represents "approximately an average of all the profiles between two piers", in the tenth edition there is shown, in its place, what the writer considers to be a more rational average profile, in that depression of the water surface does not begin until the water reaches the nose of the pier.

The value of Kutter's *n* used by the writer in Fig. 10 was 0.009, a value applicable to timber channels, well planed, and perfectly continuous.

The writer cannot agree with Mr. Horton in that Gauge No. 3 "should have been placed at the same distance up stream, or else at a greater distance than that of Bazin", on account of "velocity of approach and surface curvature". Though the writer believes that the gauge should always be placed up stream beyond the point where it would be affected by surface curvature, it must also be far enough up stream not to lie within "the angle of pressure". Fteley and Stearns* found that heads measured within this angle might exceed the true static head by as much as 2.5 times the velocity head of approach. They furthermore state:

"It is deemed reasonable and fairly in accordance with them [the experiments] to assume that the distance [upstream terminus of the angle of pressure] varies with the height of the weir above the bottom of the channel, and that it is about $2\frac{1}{2}$ times this height."

It was for this reason that the writer adopted the formula of measuring the head at a distance up stream from the weir crest (as compared with Bazin's point of measurement) proportional to the ratio of the height of the weir crest to the height of the weir used by Bazin. This same formula was used in measuring the head on one of the

* *Transactions*, Am. Soc. C. E., Vol. XII, p. 46.

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Cornell weirs, with which the writer understands Mr. Horton is thoroughly familiar. Richard R. Lyman, Assoc. M. Am. Soc. C. E., has explained this also.*

Since the presentation of the paper the writer has had occasion to perform extensive experiments in order to determine any appreciable evidence of surface curvature farther than 6 ft. from a suppressed weir, 6.56 ft. long and 3.72 ft. high. Gauges established 6 ft., and also 16.4 ft., from the weir recorded the same head for velocities of approach as high as 4 ft. per sec. (4 ft. head on the weir). As the velocities in the writer's experiments hardly exceeded this value, he is of the opinion that Gauge No. 3, established 10.625 ft. from the weir, recorded the correct head on the weir.

The writer is of the opinion that the proper procedure in investigating the effect of any single variable on an equation or phenomenon involving many variables, is either to eliminate all other variables or regulate them so that they become constants. This was precisely the writer's method in determining the relative effects of different shapes of piers; hence Mr. Molitor's incredulity in doubting that even this fact was demonstrated by the experiments seems almost inconceivable. Certainly, the only physical phenomenon which did change the size of the coefficients in the formula was the changing of the shape of the piers; and if this is not demonstrating the relative effect of the different designs, the writer is at loss to know by what name to call it.

Mr. Molitor makes the criticism that all the variables encountered in practice have not been imposed as a test for the writer's empiric constant. It is true that the effect of varying the percentage of obstruction offered by the pier was not investigated. This was unnecessary in the determination of the "effects of the different pier shapes"; but the writer even went further than this one objective, and studied the effect of the variables of depth and velocity, thus putting any bridge pier formula to a more severe test than had been applied by any experimental work to date. The writer's empiric constant withstood this test, others did not, but varied as much as 9% for any one pier. If the empiric constants for these other formulas varied this much in the consideration of only two of the variables involved, common sense dictates that the variation might even be greater when put to the test of encountering all the variables known to practice. Therefore the writer can see no grounds on which Mr. Molitor might base his "emphatic objection" to the writer's first conclusion that error exists in the present bridge pier formulas. A study of the original data for these experiments perhaps might satisfy Mr. Molitor, as he could then test the futility of obtaining an empiric constant for these existing formulas.

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* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1271.

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Although the heading-up above the pier in the writer's flume occupied a relatively greater width of the free channel than in most practical conditions, the writer cannot agree with Mr. Molitor that this phenomenon was effective in producing error in the results, as the points of measurement of static head both up stream and down stream from the pier section, were several feet remote from any local disturbance of this sort caused by the pier, and were thus statically unaffected by these disturbances.

The writer was not aware of the inconsistency in his use of symbols until Mr. Molitor brought this to his attention. It is true that there appears to be an error in the algebraic substitution on page 358, whereas if correct symbols had been used there would be none. The H_d as used on page 358 should be understood to mean the "measured head" instead of the "drop in the flume" as formerly used; and again, the H in the writer's final formula should have borne the subscript, *d*. Hence the writer's formula should not be mistaken to give the "drop in the flume" (that would be ludicrous), but gives, as he claimed, the rise or back-water caused by the pier.

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Paper No. 1410

THE SUBSIDENCE OF MUCK AND PEAT SOILS IN SOUTHERN LOUISIANA AND FLORIDA*

By Charles W. Okey, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Arthur E. Morgan, Orrin Randolph, Rudolph Hering, J. F. Coleman, William T. Lyle, William H. Kimball, S. H. McCrory, and Charles W. Okey.

Synopsis.

The object of this paper is to call the attention of engineers to the fact that, in designing drainage improvements, it is often necessary to anticipate the subsidence of muck and peat lands subsequent to drainage.

The results of some observations made in England on the subsidence of drained muck and peat lands are reviewed, and the results of first-hand observations in Louisiana and Florida are shown in detail on profiles.

It was evident that, in small districts drained by pumps, the subsidence was not a very important feature, but that in large gravity drainage districts covered with deep muck or peat, where slopes were slight and the available fall was small, subsidence would be of sufficient magnitude to make a change in design necessary.

It has long been a matter of common knowledge that swamp lands, which have soils containing a large percentage of vegetable matter, subside when drained and cultivated. In the Fenland of England such soils have been drained and cultivated for a long period, with a consequent change in surface elevation. This change has been of

* Presented at the meeting of November 7th, 1917.

such magnitude that it has made necessary the altering of a majority of the pumping plants that formerly gave adequate drainage to the lands they served. It is believed that it will be best to give the actual figures from a few representative cases rather than to state any general figures.

In the *Memoirs* of the Geological Survey of England and Wales for 1877 there appears a volume written by S. B. J. Skertchly on the "Geology of the Fenland." Mr. Skertchly mentions the general sinking of the surface on all the cultivated and drained Fens, where there is muck or peat on the surface, and gives definite figures for several locations. In 1848, an iron column, graduated in feet and inches, was sunk down through the peat, in the Middle Level of the Great Bedford Level in the vicinity of Whittlesey Mere, into the solid clay, so that the top of the column was level with the surface.* This column bears on the capital the inscription, "Level of the Ground in 1848." When Mr. Skertchly measured the subsidence of the surface on August 25th, 1870, he found it to be 7 ft. 8 in. The subsidence was measured again in 1875 and found to be 7 ft. 9 in.

Richard F. Grantham states† that in 1876 after erecting new engines and pumps, made necessary by the subsidence of the land, the total subsidence was then observed to be 8 ft. This was evidently after the new pumping plant had been put in, hence the drop of 3 in. in a year, due to the lowering of the water-table. In 1913 the total subsidence was observed by Mr. Grantham to be 10 ft. The peat at this point was originally 18 ft. thick.

Mr. Skertchly in his "Geology of the Fenland" also gives a table of the subsidence at this and other localities. This table is reproduced as Table 1.

The observers for the data in Table 1 were:

No. 1. W. H. Wheeler.
2–5. W. Wells. *Journal,* Royal Agricultural Society, Vol. XXI, 1860.
6. S. B. J. Skertchly.
7. W. Wells.
8. W. Marshall.
9. S. B. J. Skertchly.

* *Journal,* Royal Agricultural Society, Vol. XXI, 1860.
† *Proceedings,* Institution of Mechanical Engineers, London, July, 1913.

TABLE 1.—COMPRESSION OF PEAT BY DRAINAGE

No.	Locality.	Dates.	Time, in years.	Thickness, in feet.	Total compression, in inches.	Annual compression, in inches.	Total compression. Percentage.	Annual compression. Percentage.
1	East Fen..................	1806-66	60	6	24	0.4	33.3	0.55
2	Whittlesea Mere..........	1851-60	9	18	42	4.7	19.5	2.2
3	" "	do.	9	18	59	6.6	27.3	3.0
4	" "	do.	9	18	66	7.4	30.5	3.4
5	" "	do.	9	18	73	8.1	33.8	3.7
6	" "	1848-70	22	18	92	4.18	42.6	1.9
7	" "	1848-75	27	18	93	3.44	43.0	1.59
8	Hilgay....................		26	10	52	5.2	43.3	1.7
9	Wood Fen..................	1854-74	20	8	37	1.85	38.5	1.9

In commenting on Table 1, Mr. Skertchly states:

"The mean annual percentage of compression from the above data is 2.2, which under similar circumstances may be taken as the rate for a long series of years. By 'similar circumstances' is meant low-lying peat in which very little fall can be obtained for the drains; for it is clear that if drains could be cut deep into or through the peat the compression would be more rapid."

Swamp soils which are largely of vegetable origin are classed as cumulose by the United States Bureau of Soils, with the subdivisions of peat, peaty muck, and muck. Peat is composed almost entirely of partly decomposed vegetable matter; muck has a considerable percentage of silt, clay, and sand mixed with well-decayed and finely-divided vegetable matter. The different types merge into each other. Although it is not certain that all the soils considered in this paper would be properly classed as muck soils, the term will be used, because it will probably fit most of the cases and because it is in common usage in Southern Louisiana and Florida. The purpose of this paper is not to classify the soils examined, but to give a recital of facts observed.

The writer has been engaged for the past 7 years in making investigations of the drainage of muck soils in Southern Louisiana and for the past 2 years has had charge of similar work in Florida. This work was carried on for the U. S. Department of Agriculture, Office of Public Roads and Rural Engineering, Division of Drainage Investigations, and was under the immediate supervision of the Division Chief, S. H. McCrory, M. Am. Soc. C. E., under the general direction of L. W. Page, M. Am. Soc. C. E., Director of the Bureau.

The subsidence of such soils is due to three main causes: drying, decay, and cultivation. Shrinkage and the consequent subsidence, due to drying, affects most the top layer, but it extends in a decreasing rate as deep as the soil is drained. Tests have shown that undrained Louisiana muck shrinks about 60% in volume when completely dry, and that it will regain only 70% of its original volume when saturated for a long time. Therefore, long-continued dry weather and deep drainage cause a shrinkage in the deeper layers of muck which is not counterbalanced by an increase in volume due to subsequent precipitation, or rising of the water-table. Only such part of the muck as is always saturated is entirely free from shrinkage due to drying.

The vegetable material in muck soils exists in a state of partial decay. In the undrained state it is saturated with water to such an extent that the process of decay is relatively slow. After drainage the air enters, and decay is much more rapid. The warm, humid climate of Southern Louisiana and Florida is very favorable to the rapid decay of vegetable material, much more so than in places where the surface is frozen for a part of the year. Like the effect of drying, that of decay is greatest in the top layer of material; but examination has shown that, after some years of drainage, the character of the muck at a depth of 2 ft. had been materially changed from its condition prior to drainage. Although the effect of decay is not as rapid in action as that due to drying, it is practically continuous. The complete decay of the vegetable material causes some loss of weight and a considerable loss in volume, thus gradually reducing the surface elevation of the muck.

As the effect of both the foregoing agencies is greatest in the top layer of the muck, the density of this top layer is gradually increased. After this comparatively dense material attains a thickness of a few inches, it prevents free circulation of the air into the muck below, and then drying and decay are much slower in their action. Eventually, this layer attains such a thickness that further subsidence of the surface is scarcely noticeable.

Cultivation increases the subsidence directly, by the mechanical effect of weight compacting the soil, and indirectly, by increasing the action of drying and decay. Muck soils which are so soft after drainage that they will not permit of the use of farm animals and machinery are compacted from 4 to 6 in. by the first plowing. This

first plowing is usually done with some form of tractor, with broad wheels which cover practically all the surface plowed. Subsequently, when the muck is cultivated with farm animals and machinery, the surface receives unit pressures far greater than those exerted by the broad-wheeled tractor, and a further compacting results. The underlying material, which is turned to the surface by plowing, is exposed to a greater drying action than would otherwise result. Decay, also, is hastened in the material thus brought to the surface. It is the experience in cultivating newly-reclaimed muck soils that, for a number of years after the first cultivation, a uniform depth of plowing will bring to the surface each year a considerable layer of muck which was undisturbed by the previous year's plowing. This layer of new material gradually decreases in thickness, as the thickness of the layer of plowed material gradually increases. Finally, the cultivated layer attains such a density that the combined forces of drying, decay, and compacting reduce its thickness very little. If the land is not plowed deeper than this layer, the subsidence of the remaining muck is very slow; but, if the land is plowed deep enough to reach undisturbed muck, further subsidence results.

The rate at which these agencies cause muck soils to subside has been uncertain, and it was with the idea of getting definite information that the writer was authorized to make comprehensive field investigations. The work was undertaken in Southern Louisiana in the spring of 1915, and in Florida in the early spring of 1916. The field-work in Louisiana was done by the writer, and the work in Florida by Mr. F. E. Staebner under the direction of the writer. In all, 22 districts were examined. The investigations covered muck ranging in depth from a few inches to 16 ft.; it included undrained muck, that which had been under cultivation for 17 years, and that which had such a large percentage of vegetable material that it would perhaps properly be classed as peat; it also included other muck which contained a large percentage of river silt. Profiles were run on the drained and undrained muck. Samples of muck were taken, and the weight of the dry material, per cubic foot, was determined. An attempt has been made to place all the information gathered on the profiles, Figs. 1 to 13, in such a manner that the details for each district can thus be more readily grasped than from lengthy descriptions.

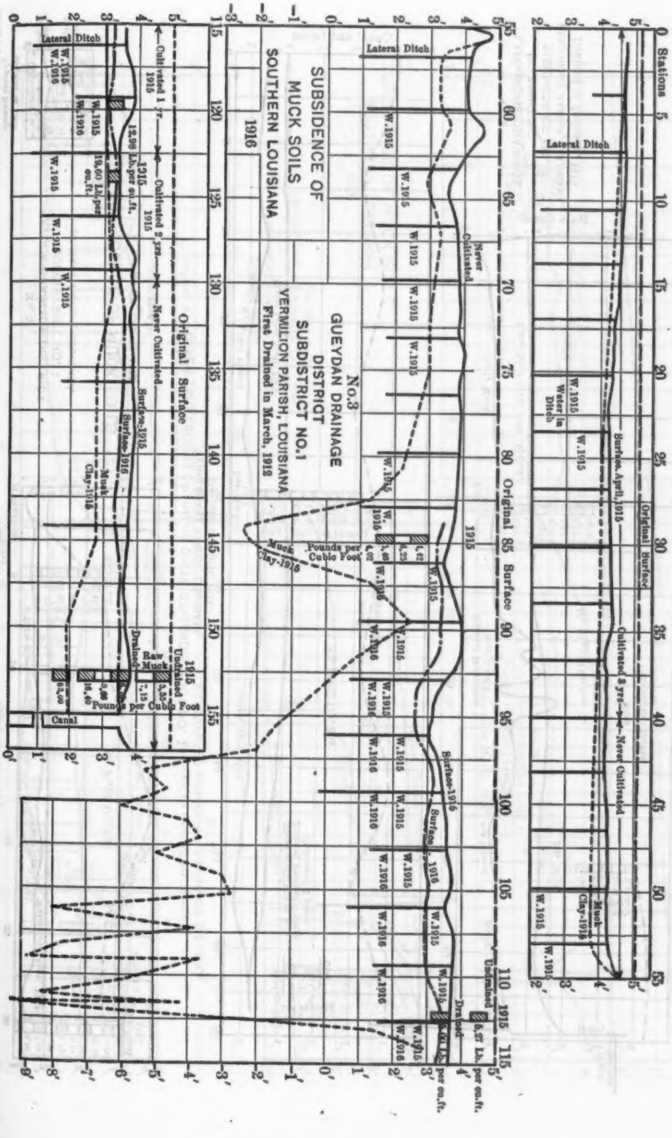

SUBSIDENCE OF
MUCK SOILS
SOUTHERN LOUISIANA
1916
No.3
GUEYDAN DRAINAGE
DISTRICT
SUBDISTRICT NO.1
VERMILION PARISH, LOUISIANA
First Drained in March, 1912
Stations
Lateral Ditch
Original Surface
Surface-April, 1915
Cultivated 3 yrs.
Never Cultivated
Muck
Clay-1916
Water in Ditch
Never Cultivated
Surface-1916
Muck
Clay-1916
Pounds per Cubic Foot
Undrained
Drained
Cultivated 1 yr. 1915
Cultivated 2 yrs. 1915
Surface-1915
Surface-1916
Raw Muck
Canal
Pounds per Cubic Foot

FIG. 1.

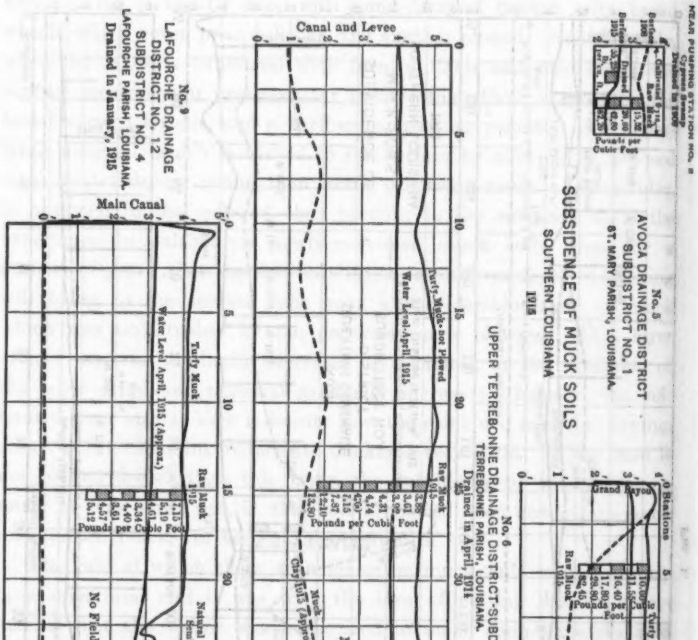

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1915
No. 5
AVOCA DRAINAGE DISTRICT
SUBDISTRICT NO. 1
ST. MARY PARISH, LOUISIANA.
NEAR PUMPING STATION NO. 2
Cypress Swamp
Drained in 1909
Pounds per Cubic Foot
NEAR PUMPING STATION NO. 2
Grass covered Prairie
Drained in Jan., 1915
Grand Bayou
Stations
Turfy Muck-not Plowed
No Field Ditches
Surface-April, 1915
Water Level-April, 1915
Raw Muck 1915
Pounds per Cubic Foot
No. 6
UPPER TERREBONNE DRAINAGE DISTRICT-SUBDISTRICT NO. 1
TERREBONNE PARISH, LOUISIANA.
Drained in April, 1914
Canal and Levee
Turfy Muck-not Plowed
Water Level-April, 1915
Surface-April, 1915
No Field Ditches
Pounds per Cubic Foot
Surface-1915
Clay
Pounds per Cubic Foot
No. 9
LAFOURCHE DRAINAGE
DISTRICT NO. 12
SUBDISTRICT NO. 4
LAFOURCHE PARISH, LOUISIANA.
Drained in January, 1915
Main Canal
Turfy Muck
Water Level April, 1915 (Approx.)
Raw Muck 1915
Pounds per Cubic Foot
Natural Depression Semi-Liquid
No Field Ditches
Turfy Muck-not Plowed
Surface-April, 1915
Raw Muck 1915
Pounds per Cubic Foot
Canal
Levee
Water Surface

FIG. 2.

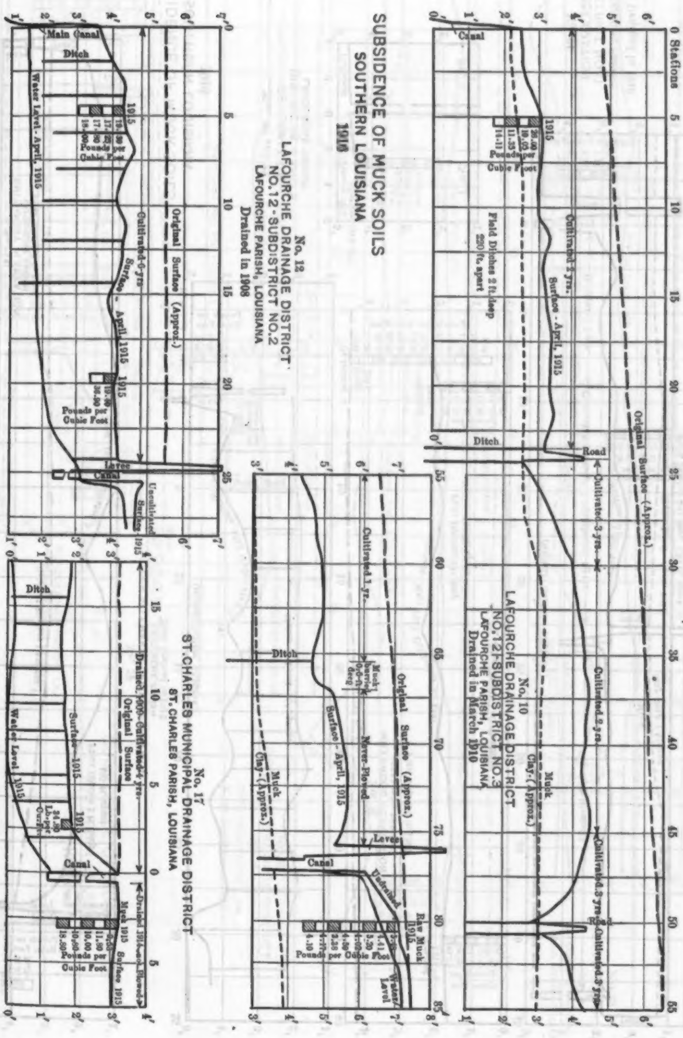
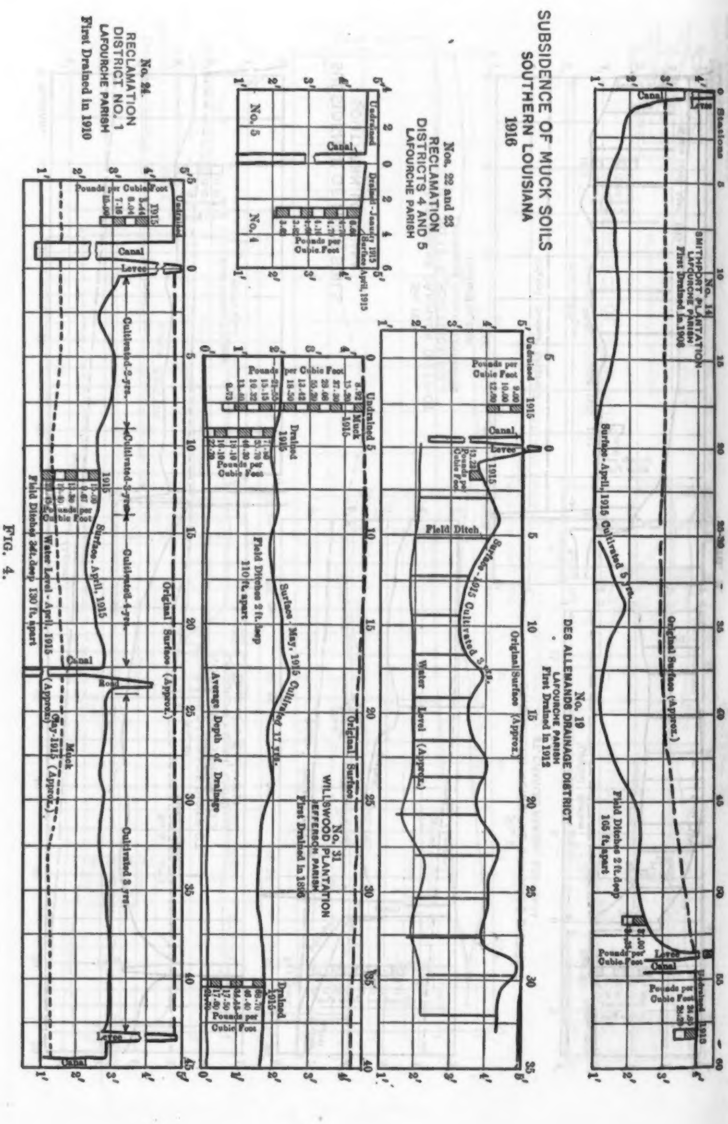

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No. 10
LAFOURCHE DRAINAGE DISTRICT
NO. 12 SUBDISTRICT NO. 3
LAFOURCHE PARISH, LOUISIANA
Drained in March 1910
No. 12
LAFOURCHE DRAINAGE DISTRICT
NO. 12-SUBDISTRICT NO. 2
LAFOURCHE PARISH, LOUISIANA
Drained in 1908
No. 17
ST. CHARLES MUNICIPAL DRAINAGE DISTRICT
ST. CHARLES PARISH, LOUISIANA
Original Surface (Approx.)
Surface - April, 1915
Water Level - April, 1915
Pounds per Cubic Foot
Field Ditches 2 ft. deep 220 ft. apart
Clay-(Approx.)
Muck
Main Canal
Ditch
Canal
Levee
Road
Stations

FIG. 3.



SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No. 14
SMITHPORT PLANTATION
LAFOURCHE PARISH
First Drained in 1908
Surface-April, 1915 Cultivated 5 yrs.
Original Surface (Approx.)
Field Ditches 2 ft. deep
105 ft. apart
Pounds per Cubic Foot
No. 19
DES ALLEMANDS DRAINAGE DISTRICT
LAFOURCHE PARISH
First Drained in 1912
Field Ditch
Water Level (Approx.)
Nos. 22 and 23
RECLAMATION
DISTRICTS 4 AND 5
LAFOURCHE PARISH
No. 31
WILLSWOOD PLANTATION
JEFFERSON PARISH
First Drained in 1896
Surface, May, 1915 Cultivated 17 yrs.
Average Depth of Drainage
Field Ditches 2 ft. deep
110 ft. apart
No. 24
RECLAMATION
DISTRICT NO. 1
LAFOURCHE PARISH
First Drained in 1910
Surface-April, 1915
Field Ditches 3 ft. deep 130 ft. apart
Canal
Levee
Road
Muck

FIG. 4.

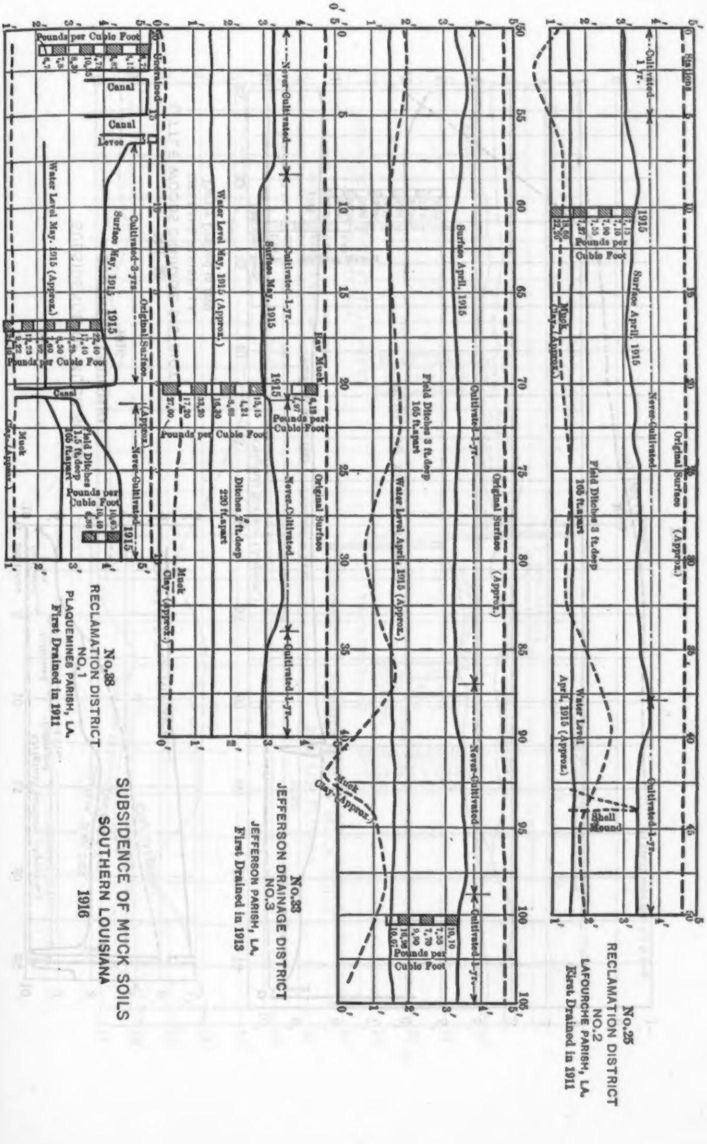

No.25
RECLAMATION DISTRICT
NO.2
LAFOURCHE PARISH, LA.
First Drained in 1911
No.33
JEFFERSON DRAINAGE DISTRICT
NO.3
JEFFERSON PARISH, LA.
First Drained in 1913
No.38
RECLAMATION DISTRICT
NO.1
PLAQUEMINES PARISH, LA.
First Drained in 1911
SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916

FIG. 5.

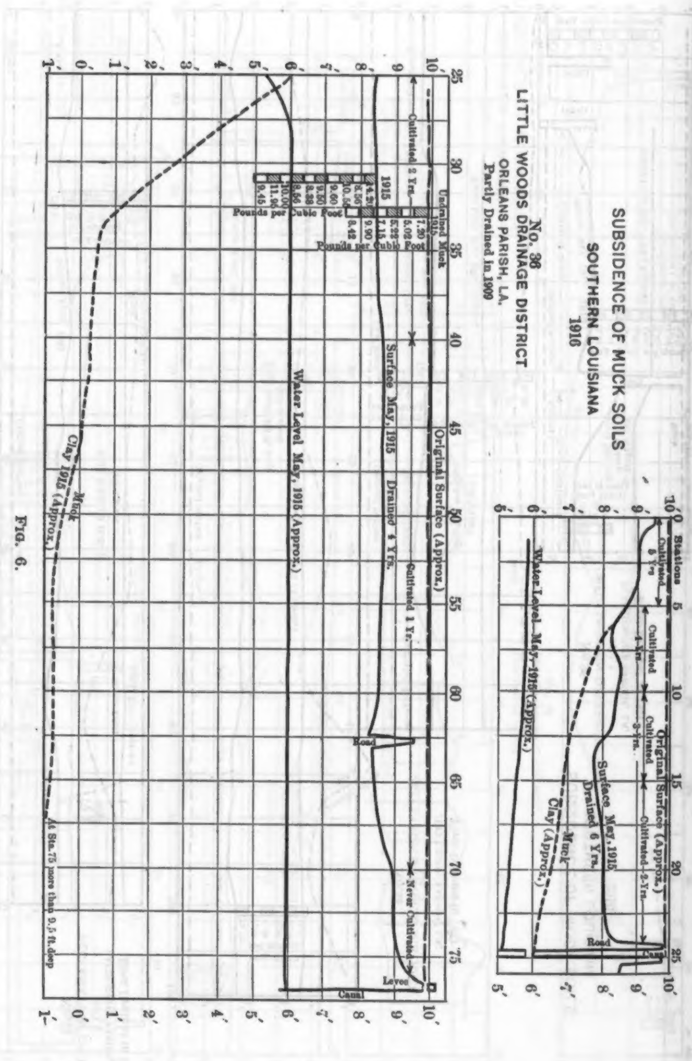

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No. 36
LITTLE WOODS DRAINAGE DISTRICT
ORLEANS PARISH, LA.
Partly Drained in 1909
Original Surface (Approx.)
Surface May, 1915
Water Level May, 1915 (Approx.)
Clay 1915 (Approx.)
Muck
Road
Canal
Levee

FIG. 6.

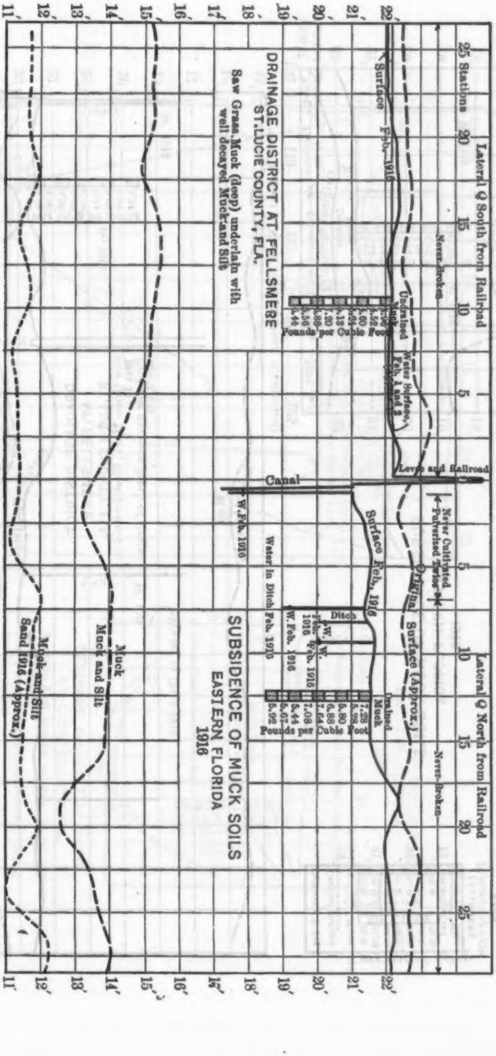

SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
DRAINAGE DISTRICT AT FELLSMERE
ST. LUCIE COUNTY, FLA.
Saw Grass Muck (deep) underlain with well decayed Muck and Silt
Lateral Q South from Railroad
Lateral Q North from Railroad
Stations
Never-Broken
Never Cultivated Pulverized Twice
Original Surface (Approx.)
Levee and Railroad
Canal
Ditch
Surface Feb. 1916
Water in Ditch Feb. 1916
Water Surface, Feb. 1 and 2
Undrained Muck
Drained Muck
Pounds per Cubic Foot
Muck
Muck and Silt
Sand 1916 (Approx.)

FIG. 7.

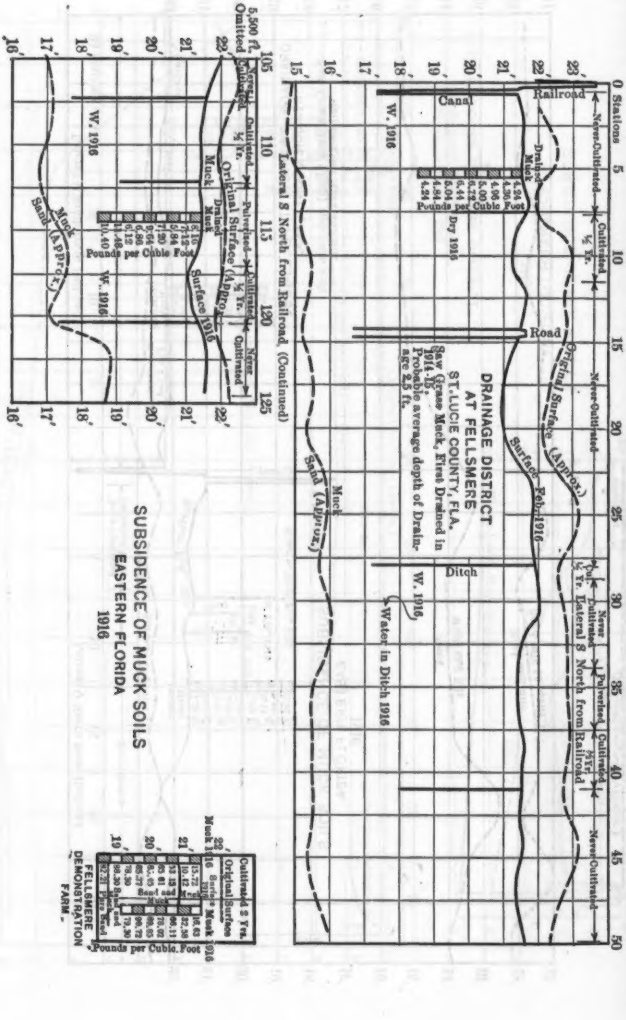

DRAINAGE DISTRICT
AT FELLSMERE
ST.LUCIE COUNTY, FLA.
Saw Grass Muck, First Drained in 1914-15.
Probable average depth of Drainage 2.5 ft.
Lateral 8 North from Railroad, (Continued)
SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
FELLSMERE
DEMONSTRATION
FARM
Pounds per Cubic Foot

FIG. 8.

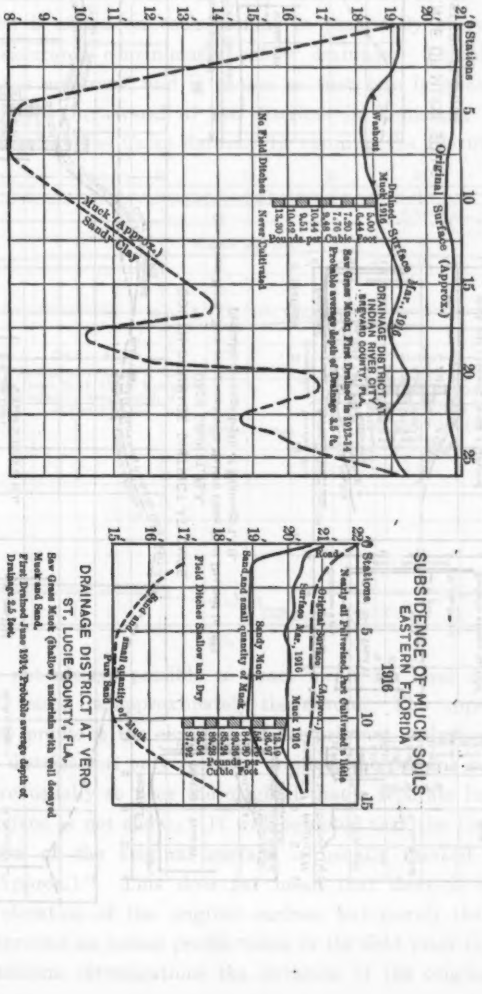

SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
Stations
Original Surface (Approx.)
Drained—Surface Mar., 1916
Muck 1916
Washout
DRAINAGE DISTRICT AT
INDIAN RIVER CITY
BREVARD COUNTY, FLA.
Saw Grass Muck; First Drained in 1912-14
Probable average depth of Drainage 2.5 ft.
Pounds per Cubic Foot
No Field Ditches Never Cultivated
Muck (Approx.)
Sandy Clay
Road
Nearly all Pulverized, part Cultivated a little
Original Surface (Approx.)
Surface Mar., 1916
Muck 1916
Sand and small quantity of Muck
Sandy Muck
Field Ditches Shallow and Dry
Sand and small quantity of Muck
Pure Sand
DRAINAGE DISTRICT AT VERO
ST. LUCIE COUNTY, FLA.
Saw Grass Muck (shallow) underlain with well decayed Muck and Sand.
First Drained June 1914, Probable average depth of Drainage 2.5 feet.

FIG. 9.

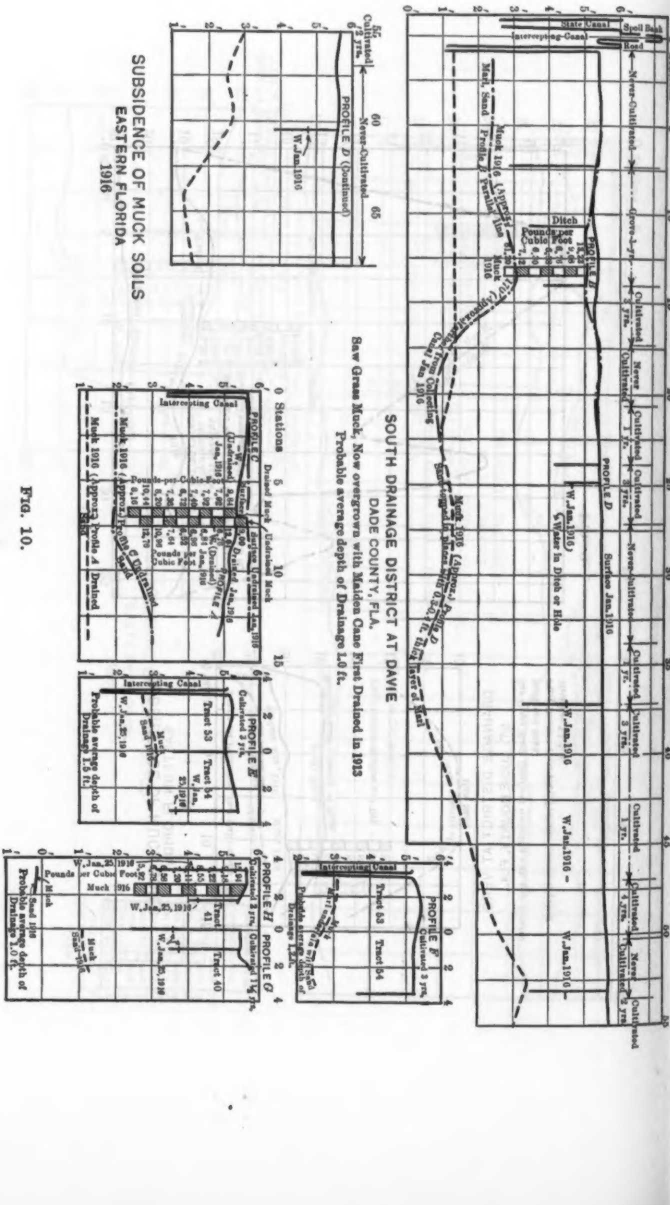

SOUTH DRAINAGE DISTRICT AT DAVIE
DADE COUNTY, FLA.
Saw Grass Muck, Now overgrown with Maiden Cane First Drained in 1913
Probable average depth of Drainage 1.0 ft.
SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916

FIG. 10.

The number over the title of each profile on Figs. 1 to 6 corresponds with the number on the map of Southern Louisiana, Plate V, showing the reclamation districts. The title of the Florida districts, Figs. 14 to 18, gives the number and name of the district, and the county in which it is situated. The date on which the land was first drained is stated on each profile; in practically every case the land was saturated continuously before drainage. The datum for each profile is arbitrary, and is chosen so that zero is below any elevation platted. On several of the districts in Louisiana the levels were referenced to the Cairo datum. In running the Florida profiles,

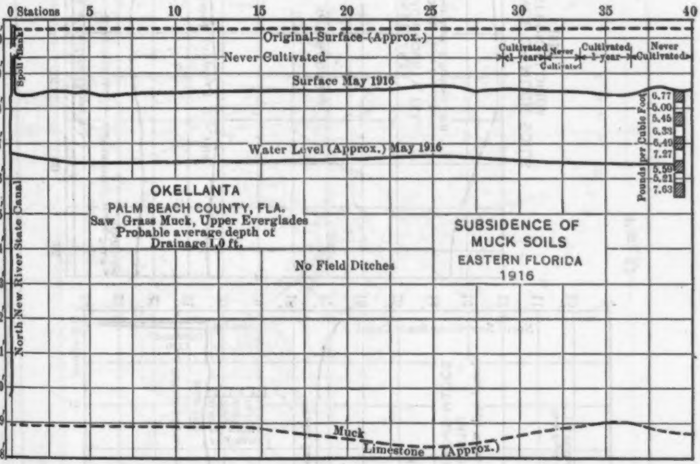


FIG. 11.

access was not always possible to exact mean sea level datum, but that, in all cases, is approximately the datum. The upper line on nearly every profile is the elevation of the original undrained surface. Where the district was only recently drained, the present surface elevations were usually so near the original that a separate line for the original surface is not shown. It will be noted that the line showing the elevation of the original surface is usually marked "Original Surface (Approx.)." This does not mean that there is any doubt about the elevation of the original surface, but merely that the line does not represent an actual profile taken in the field prior to drainage. In the Louisiana investigations the elevation of the original surface

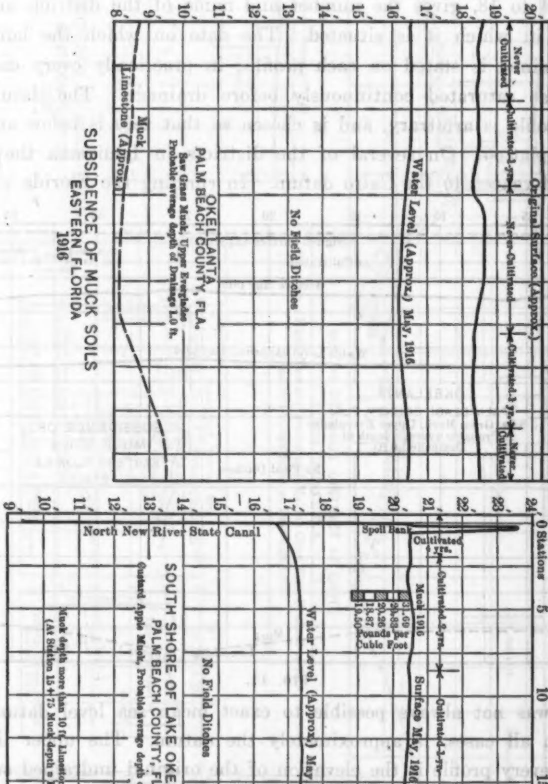

40 Stations 45 50 55 60 65
Never Cultivated
Cultivated 3 yrs.
Original Surface (Approx.)
Never Cultivated
Cultivated 3 yrs.
Never Cultivated
Water Level (Approx.) May, 1916
No Field Ditches
OKEELANTA,
PALM BEACH COUNTY, FLA.
Saw Grass Muck, Upper Everglades
Probable average depth of Drainage 1.0 ft.
Muck
Limestone (Approx.)
SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
0 Stations 5 10 15 20
Spoil Bank
Cultivated 4 yrs.
Cultivated 3 yrs.
Cultivated 1 yr.
Never Cultivated
Cultivated 4 yrs.
Never Cultivated
Lagoon
Muck 1916
Surface May, 1916
Pounds per Cubic Foot
Water Level (Approx.) May 1916
North New River State Canal
No Field Ditches
SOUTH SHORE OF LAKE OKEECHOBEE
PALM BEACH COUNTY, FLA.
Custard Apple Muck, Probable average depth of Drainage 2.0 ft.

FIG. 12.

RECLAMATION DISTRICTS

NO.	PARISH	NAME OF DISTRICT	ACRES	REMARKS
1	Calcasieu	Reclamation District No. 1. Lake Charles	700	
2	Cameron	Cameron Drainage District No. 2	14,730	
*3	Vermilion	Reclamation District No. 1 & 2, Florence	7,400	
4	St. Mary	St. Mary Drainage District No. 2	19,000	
5	St. Mary	Avoca Drainage District	15,600	
6	Terrebonne	Upper Terrebonne Drainage District	4,240	
7	Lafourche	Reclamation District No. 7, Raceland	3,840	
8	Lafourche	Reclamation District No. 6, Raceland	4,760	
9	Lafourche	Reclamation District No. 4, Raceland	4,240	
*10	Lafourche	Reclamation District No. 3, Raceland	2,400	
11	Lafourche	Reclamation District No. 1, Raceland	800	300 acres in cultivation with 1,360 acres being added to district.
12	Lafourche	Reclamation District No. 2, Raceland	2,300	
13	Lafourche	Reclamation District No. 5, Raceland	1,320	
*14	Lafourche	Smithport Plantation	647	
15	Lafourche	Fay Plantation	2,000	300 acres in cultivation District to be enlarged
16	St. Charles	St. Charles Municipal Drainage Dist. No. 2	7,130	
17	St. Charles	St. Charles Municipal Drainage Dist. No. 1	9,860	Contains Sub-District of 800 acres in cultivation.
18	St. Charles	St. Charles Drainage District No. 1	2,540	
*19	Lafourche	Des Allemands Drainage District No. 1 & 2	3,320	1,880 acres under pump 1,000 acres being added
20	Lafourche	Des Allemands Drainage District No. 3	5,400	
21	Lafourche	Gheens Plantation	7,600	
22	Lafourche	Reclamation District No. 5, Larose	2,700	
23	Lafourche	Reclamation District No. 4, Larose	2,560	
24	Lafourche	Reclamation District No. 1, Larose	840	
25	Lafourche	Reclamation District No. 2, Larose	2,730	
26	Lafourche	Reclamation District No. 3, Larose	2,560	
27	Lafourche	Reclamation District No. 2, Golden Meadows	1,200	
28	Lafourche	Reclamation District No. 1, Golden Meadows	1,780	
29	St. Charles	La Branche Drainage District	7,500	
30	Jefferson	Kenner Drainage District	6,300	
*31	Jefferson	Willswood Plantation	3,000	
32	St. Charles	Lake Catsouache Reclamation District	5,000	
*33	Jefferson	Jefferson Drainage District No. 2	5,000	
34	Jefferson	Jefferson Drainage District No. 3	5,100	
*35	Orleans	New Orleans Land Co. District No. 1	1,330	
*36	Orleans	Little Woods District	7,500	
37	Plaquemines Jefferson	Plaquemines-Jefferson Drainage District	37,750	
*38	Plaquemines	Reclamation District No. 1, Poydras	2,500	
39	St. Bernard	Reclamation District No. 2, Poydras	15,000	
40	St. Bernard	Shell Beach Drainage District	7,000	
41	St. Bernard	Gillem Plantation	3,000	
42	St. Bernard	Bayou La Loutre Drainage District	7,000	
43	Plaquemines	Venice Drainage District	1,100	

*Districts in which records of rainfall and pumping are being kept by U.S. Dept. of Agriculture

NAME OF DISTRICT	ACRES
…amation District No. 4, Larose	2,560
…mation District No. 1, Larose	640
…mation District No. 2, Larose	2,720
…mation District No. 3, Larose	2,560
…mation District No. 2, Golden Meadows	1,200
…mation District No. 1, Golden Meadows	1,780
…rauche Drainage District	7,500
…er Drainage District	6,300
…wood Plantation	2,600
… Cataouache Reclamation District	5,000
…rson Drainage District No. 2	5,000
…rson Drainage District No. 3	5,100
… Orleans Land Co. District No. 1	1,380
… Woods District	7,500
…uemines-Jefferson Drainage District	37,750
…amation District No. 1, Poydras	2,500
…amation District No. 2, Poydras	15,000
… Beach Drainage District	7,950
…n Plantation	3,800
…u La Loutre Drainage District	7,000
…e Drainage District	1,100

PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1410.
OKEY ON
SUBSIDENCE OF MUCK AND PEAT SOILS.

was usually determined by a profile taken on the undrained land parallel with the one on the drained land and as close as possible to it. Due to the uniformity in the elevations of the land, it is considered that the error introduced by this method would not be greater than 0.2 ft., except for small surface inequalities. Practically all the muck lands of Southern Louisiana are only a foot or two above sea level, and, as shown by the profiles, are almost flat. Before drainage the lands are saturated continuously and usually flooded several inches

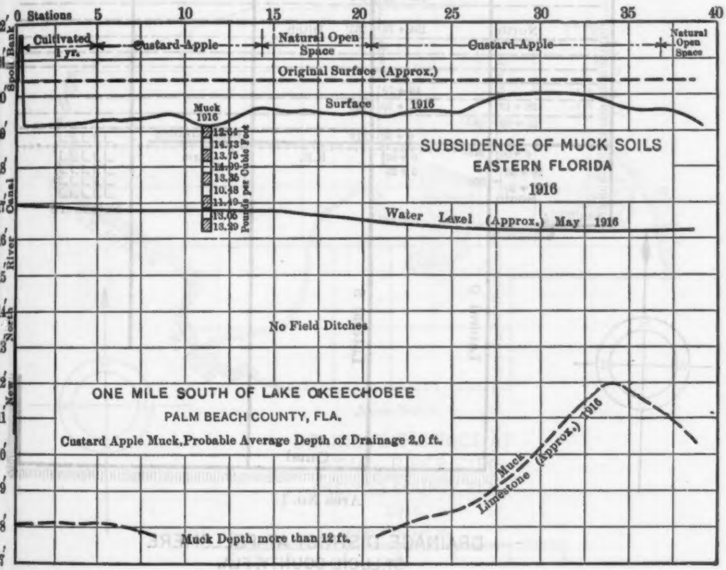


FIG. 13.

deep. The elevation of the undrained land around a district could change but very little, except for a slight subsidence due to the slight surface drainage furnished by the canals along the levees in the drained district. Any change that did occur would make the apparent subsidence of the drained land less than it really was. In running the profiles on both the drained and the undrained lands, the elevations on lines perpendicular to the profile were taken at frequent intervals, which showed that the land was level crosswise to the profiles. Where the profile is marked "Original Surface", data were available which

were taken before the land was drained. Although the elevation of the lands examined in Florida is considerable, such lands, also, are almost flat. In nearly every case the "Original Surface (Approx.)" is platted from profiles run on the center line of canals since constructed. As shown on Figs. 14 to 18, the profiles were then run parallel to these canals and a short distance from them. The same precautions as in

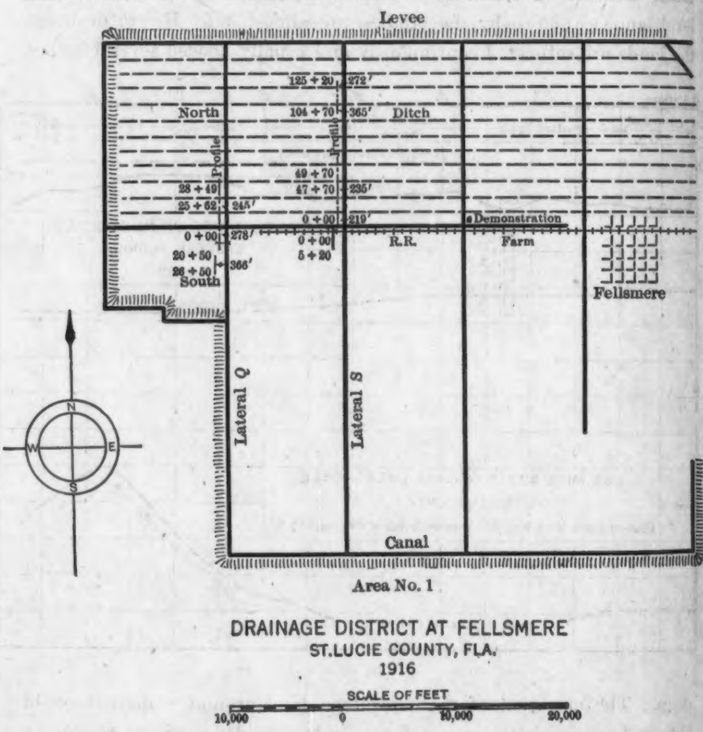


FIG. 14.

the work in Louisiana were taken in order to make sure that the land did not slope crosswise to the line of profile.

The second line from the top shows the surface elevation at the time the profiles were run. Notes indicate how long the surface had been cultivated at that time. The depth and spacing of the field ditches are shown on some of the profiles by vertical lines drawn to

scale; on others the depth and spacing are indicated by a note. The small maps, Figs. 14 to 18, show the direction of the small field ditches with respect to the profile. Usually, the surface near the deeper ditches and canals is lower, because that portion has been drained deeper and longer and has been cultivated for a longer time. This is especially noticeable on District No. 10 (Fig. 3). Here the large collecting ditches are at intervals of about half a mile. From the first they have been kept at a depth of about 3 ft. The small field ditches

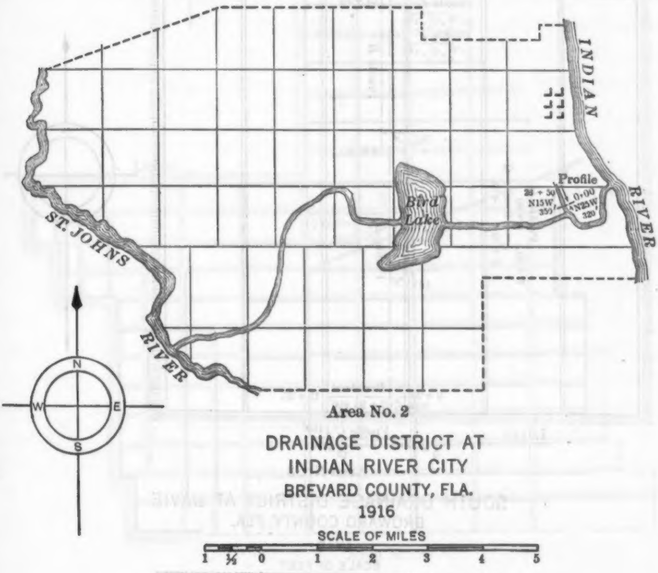


FIG. 15.

between these collecting ditches have been in very poor condition most of the time since the first drainage, the average depth of drainage being about 1 ft. until the latter part of 1914. However, when it was decided to cultivate the land back from the collecting ditches, the field ditches were cleaned out to a depth of nearly 3 ft. After a few years more of cultivation this difference in subsidence in this particular district will be less noticeable, and will eventually disappear.

The approximate location of the water-table is shown by another line. As the 2 months previous to the field examinations were with-

out rain, the water-table was uniformly lower than usual. Where this line is marked "Average depth of drainage" it refers to the average condition, rather than to the particular one which existed when the examinations were made. The standard depth for field ditches in these lands has been 3 ft. However, the ditches deteriorate very rapidly, and their average depth has been about 2 ft. In most of the districts the average depth of drainage has been not more than 2 ft.,

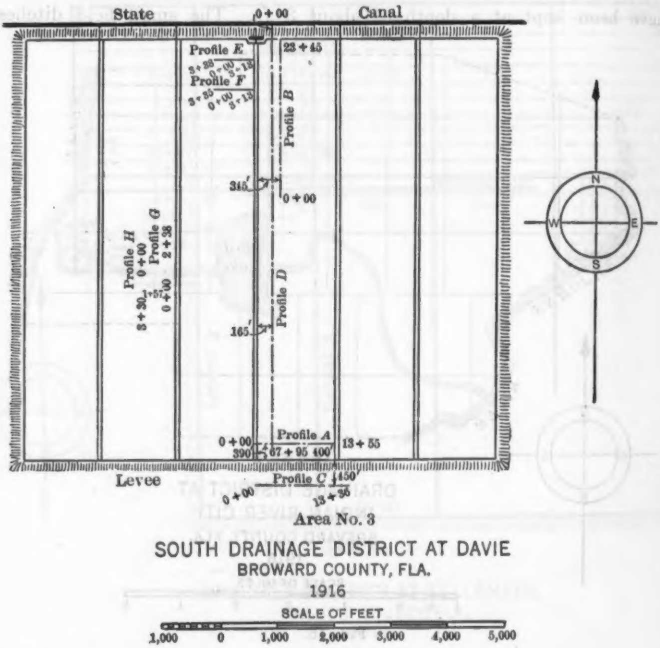


FIG. 16.

and for a large portion of the time it was only 1 ft. As the surface subsided the ditches had to be continually deepened, in order to afford even the shallow depth of drainage just mentioned. As a general proposition, it might be stated that the lands on which the profiles were run suffered from too shallow drainage during a portion of the time since they were first drained, and the water-table was probably never lowered farther below the surface than would be considered good drainage practice.

The approximate boundary between the muck and the underlying strata is shown by still another line. Borings and excavations were made at frequent intervals, but there was no attempt to get a com-

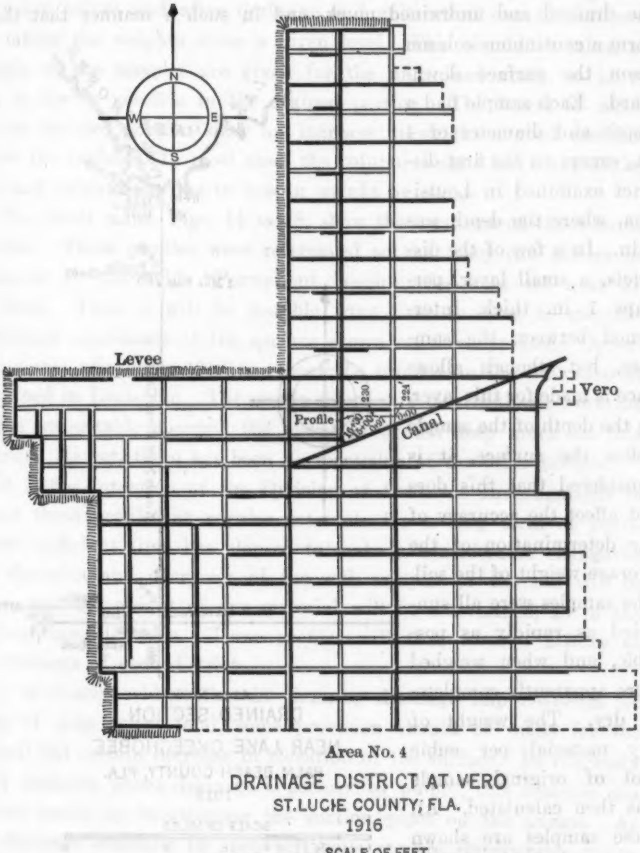


Fig. 17.

plete and accurate profile on the bottom of the muck. Quite often, the muck merges gradually into the silty clay or sand, so that the boundary between the two can be determined only approximately. The error in the location of this line is probably not more than 0.5 ft.

Where the districts had been cultivated so long that the muck had practically disappeared, this line is omitted from the profile.

Samples of soil were taken to as great a depth as possible, in both the drained and undrained muck, and in such a manner that they form a continuous column from the surface downward. Each sample had a depth and diameter of 4 in., except on the first district examined in Louisiana, where the depth was 6 in. In a few of the districts, a small layer, perhaps 1 in. thick, intervened between the samples, but, though allowance is made for this layer, in the depth of the sample below the surface, it is considered that this does not affect the accuracy of the determination of the average weight of the soil. The samples were all sundried as rapidly as possible, and when weighed were apparently completely dry. The weight of dry material per cubic foot of original sample was then calculated. All these samples are shown on the profiles at their true vertical scale and location, with the weight of dry material per cubic foot. By adding the weights of the various layers and correcting for the fact that the thickness of each is not a foot, the total weight of a section of a given depth can be obtained. Especial attention is called to the weight per cubic foot

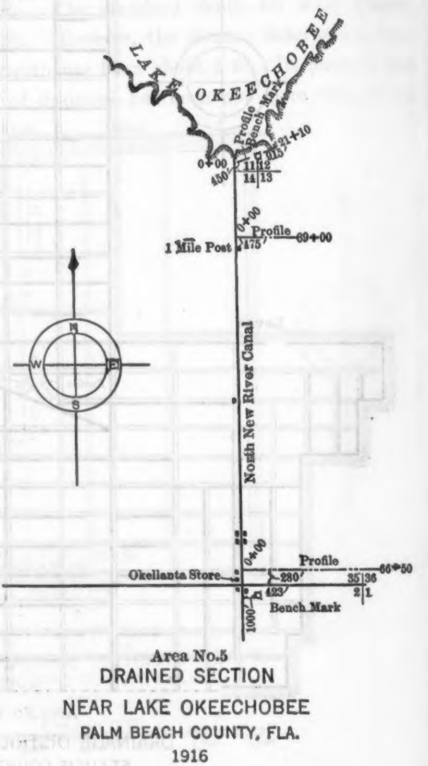


FIG. 18.

of the top layer of drained and cultivated muck. The soils which have been cultivated longest are the heaviest. The weight of a column of muck before and after drainage and cultivation can be obtained by taking the weights above a given level. This should be taken as deeply as the samples are given for the undrained muck, so as to get as low as possible in the drained portion. The layers of muck in the drained portion show an increase in weight a good distance below the surface. In most cases the column weighs less after drainage and cultivation, due to loss in weight by decay.

The small maps, Figs. 14 to 18, show the locations of the Florida profiles. These profiles were referenced so that they can always be relocated in the field. Permanent bench-marks were set near all of them. Thus it will be possible from time to time to measure additional subsidence of the surface shown on these profiles. Opportunity was offered in 1916 to repeat the profiles on the first district examined in Louisiana. The profile shows very clearly that a lowering of the water-table changed the elevation of the deep muck on this district. As yet, there has been no repetition of the other profiles.

It is the intention of the Division of Drainage Investigations to repeat these profiles at regular intervals for a long term of years. Where sufficient time has elapsed to make a considerable change in the elevation and character of the soil, samples of the soil will be taken, and the weight of dry material will be determined, as in the original investigations. These profiles will thus form a history of the drainage of each district.

It is clearly evident that in planning drainage improvements for areas of deep muck land, some provision should be made for the gradual but certain decrease in elevation of the surface. In relatively small districts, where drainage is secured by pumps, this decrease can be met easily by lengthening the suction pipes on the pumps. As the drainage channels in such soft soils require considerable maintenance in the earlier years of drainage, they can be deepened accordingly. Where the land is drained by gravity, the elevation of the water at the outlet is usually fixed, and a change in elevation of the land to be drained will mean a revision of the hydraulic gradient in the main drainage channels, with the consequent change in width and depth of the channels.

DISCUSSION

Mr. Morgan.

ARTHUR E. MORGAN,* M. AM. SOC. C. E. (by letter).—As there are several million acres of muck and peat lands in the United States, any information relative to their character is of material importance. Mr. Okey's paper is one of comparatively few contributions of definite value on this subject which have been made in the United States. It indicates conclusively that any project for the reclamation of muck or peat lands which ignores the probability of very great soil settlement has omitted a vital factor in the problem.

Although Mr. Okey's studies have established the fact of very great subsidence in drained muck soils, yet few, if any, of his measurements were made in those soils where the greatest subsidence would take place. Most of his measurements in Southern Louisiana were of necessity on soils which, for a long time, have been subject to occasional overflow by the Mississippi and have received more or less silt deposit. The muck a mile from Lake Okeechobee and that near Davie, in Florida, is already partly decomposed, and, therefore, a certain amount of compacting had taken place long before the first records of ground surface elevations were made. The great interior of the Everglades is composed of a coarse brown fibrous peat, in which even a materially greater subsidence may be expected than in most of the cases recorded by Mr. Okey. Tests by burning show the ash content of Louisiana muck south of New Orleans to be from 20 to 50% of the dry weight of the raw muck. The ash content of the interior of the Everglades, as recorded by the United States Soil Survey, is from 3 to 6%, according to the writer's remembrance.

Engineers of the Morgan Engineering Company recently spent about 6 months in making a detailed examination of more than 150 tracts of muck lands in various parts of the United States, principally in the Atlantic Coast States. It appears that the terms "peat" and "muck" are used to cover a wide variety of soils and soil conditions. In New York, Pennsylvania, and the adjoining States, there are considerable areas of muck lands formerly covered with dense growths of hardwood and cedar. This soil is well decomposed. When first drained, it is brown, but, in a year or two, it turns black and has the granular appearance of old-fashioned gunpowder. When cleared and put in cultivation this is the finest truck garden soil to be found in the United States, and such muck areas commonly sell for from two to five times as much per acre as the adjoining loam soils. The high value is due not primarily to great fertility, but to the exceptional ease with which such land may be tilled, to its unusual capacity for retaining capillary water, and to its loose texture, which makes perfect root development possible.

* Dayton, Ohio.

Mr. Morgan.

At the other extreme of soils of this class is the coarse brown fibrous peat, great areas of which occur in Minnesota, Wisconsin, and Florida, as well as in Canada and many parts of Europe. Nowhere does one find records of the profitable use of this soil for cultivation, except in rare cases after long and painstaking effort for its reduction. Most of this worthless brown peat occurs in open marshes, though such marshes also have produced some excellent muck. Between the valuable muck lands of the hardwood forests and the worthless brown peat of the open marshes there is every degree of composition and every condition of decay and settlement. Similarly, as to the content of mineral matter, there is every degree of variation, from the peat of the Everglades and of Minnesota wire-grass marshes, which on burning leave only 3% of ash, to muck areas so impregnated with silt by the frequent overflow of muddy water that the soil on burning loses only one-third or one-fourth of its weight. Naturally, the mineral content affects directly the possibility of subsidence. Mr. Okey's paper would have been of still greater value if he had recorded the percentage of ash in the various classes of muck investigated.

Perhaps no other soils in the United States are so imperfectly known, or vary so much between the highest agricultural value and complete worthlessness, and probably no other soils have been so exploited by promoters—sales of worthless peat being made on the reputation of valuable black muck. A continuation of this investigation, as suggested by Mr. Okey, would add greatly to the knowledge necessary for their economical development. The most significant inference from the paper is that, in consideration of the extremely flat gradients which sometimes are all that are possible, and the comparatively narrow range of fluctuation in ground-water level suitable for successful muck land agriculture in the South, all efforts to secure satisfactory gravity drainage in certain notable undertakings for the reclamation of peat and muck lands must be failures, and that the only hopeful prospect in these particular cases is for drainage by pumping.

Mr. Randolph.

ORRIN RANDOLPH,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Okey has furnished information on a subject of prime importance in connection with the drainage of muck lands, and his investigations, when completed, should make possible a more accurate forecast of the behavior of these soils after the water-table has been lowered and farming operations have been conducted.

The author has collected some of his data in Louisiana and the remainder in Florida, and the soil samples which have been examined have differed very materially, as is shown by the weights that were found. This wide difference in the nature of the material examined has produced results from which it becomes difficult to draw general conclusions.

* Lake Worth, Fla.

Mr. Randolph.

However, the observations made on the unfarmed saw grass muck lands of Florida constitute a set of experiments on soil which is comparatively uniform in character and from which certain deductions may be made.

By making use of the actual weights shown on the five Florida profiles of unfarmed saw grass muck, and reducing the weight of each foot in depth to its percentage of the total weight of the sample from which it is taken, a set of results is obtained as shown in Table 2.

An examination of Table 2, in connection with the graphic illustration, Fig. 19, shows the percentages of weights which were found.

TABLE 2.—Percentage, in Weight, of Total Sample.

Location.	1st foot.	2d foot.	3d foot.	4th foot.	5th foot.	6th foot.	7th foot.	8th foot.	9th foot.	Average weight per cubic foot of sample.
Fellsmere District Lat. Q. North...	12.8	9.3	10.2	12.1	13.4	12.4	9.5	9.9	10.4	6.33
Fellsmere District Lat. Q. South...	9.9	9.0	9.2	10.5	10.2	14.4	12.8	11.1	12.9	5.56
Fellsmere District Lat. S. North....	9.3	9.5	10.9	10.9	14.7	14.0	11.0	10.5	9.2	5.09
Indian River District.............	6.3	8.1	9.0	9.7	11.9	13.1	11.9	13.3	16.7	8.86
Upper Everglades District, Okeelanta............	12.1	8.9	9.8	11.3	11.7	13.1	10.0	9.4	13.7	6.19
Totals.............	50.4	44.8	49.1	54.5	61.9	67.0	55.2	54.2	62.9	32.03
Averages.........	10.08	8.96	9.82	10.90	12.38	13.40	11.04	10.84	12.58	6.406

In these five cases the average lowering of the water-table below the surface was 2.3 ft. The average subsidence was 1.6 ft., and the time the land had been drained, although not given in every case, would probably average about 2 years.

It would appear from the results found, and from observations of the writer, that the subsidence of saw grass muck, due to a lowering of the water-table, takes place somewhat uniformly over the entire depth of muck, and that it is not confined to that portion of the muck which stands above the water-table. It also appears that the depth of subsidence due to this cause is dependent on the depth to which the water-table has been lowered, as well as the time, within certain limits, that has elapsed after the drainage operations have become effective.

The author did not explain his method of obtaining and ascertaining the weight of the samples, and it is to be hoped that this information will be given, so that, in case independent observations are made, a basis of comparison of results may be had. Mr. Randolph.

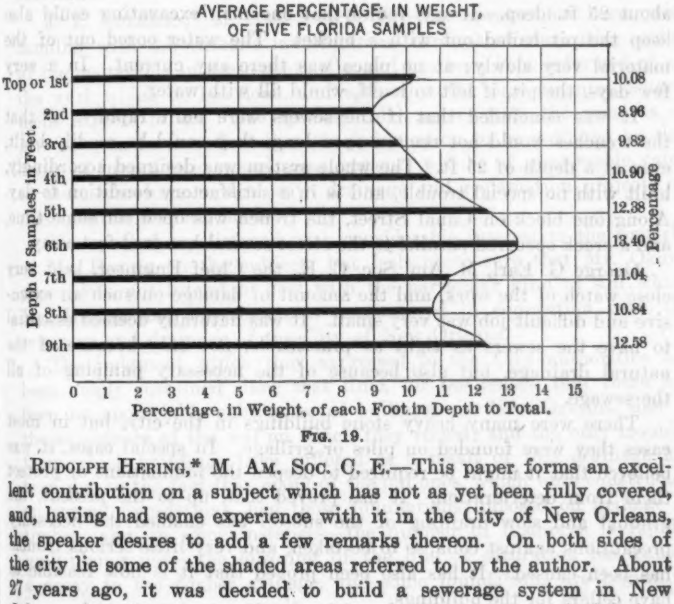


FIG. 19.

RUDOLPH HERING,* M. AM. SOC. C. E.—This paper forms an excellent contribution on a subject which has not as yet been fully covered, and, having had some experience with it in the City of New Orleans, the speaker desires to add a few remarks thereon. On both sides of the city lie some of the shaded areas referred to by the author. About 27 years ago, it was decided to build a sewerage system in New Orleans; but there was much opposition to the usual method of design. It was feared that the city buildings would collapse by the drying and settling of the soil in consequence of under-drainage. Mr. Hering.

A system was suggested by the late Col. George E. Waring, Jr., of keeping the sewers quite near the surface and pumping the sewage frequently. Another was suggested by Mr. Broughton, adopting the Shone system, which, at frequent intervals, caused the sewage to be pumped automatically by compressed air. A profile of a sewer built on such a system would look like a saw.

The speaker was engaged in 1892 to give an opinion on these propositions, somewhat novel and covering a difficult problem. It was suggested to make some excavation tests. The ground-water level in New Orleans stood quite near the surface, and the material consisted of clay and very fine sand, with some organic matter. The clay was more than half of the bulk. Consequently, it was impracticable to

* New York City.

Mr. Hering. have any cellars in the city, and, in the cemeteries, the bodies were buried in mounds above the general surface.

To drain this soil would mean shrinkage, and the question was, how much and how soon? A test pit was dug, about 5 ft. square and about 25 ft. deep. It was found that one man excavating could also keep the pit bailed out with a bucket. The water oozed out of the material very slowly; at no place was there any current. In a very few days, the pit, if left to itself, would fill with water.

It was concluded that if the sewers were built rapidly, so that the trenches would not remain open long, they could be readily built, even at a depth of 25 ft. The whole system was designed accordingly, built with no special trouble, and is in a satisfactory condition to-day. Along one block on Canal Street, the trench was open for some time, and a crack occurred parallel to the street several hundred feet away.

George G. Earl, M. Am. Soc. C. E., the Chief Engineer, kept very close watch of the work, and the amount of damage on such an extensive and difficult job was very small. It was naturally deemed essential to have the sewers as tight as practicable, not only because of the natural drainage, but also because of the necessary pumping of all the sewage.

There were many heavy stone buildings in the city, but in most cases they were founded on piles or grillage. In special cases, it was believed that it might be required to deepen the foundations or protect them from deterioration. It has proved that, up to the present, the gradual and slow draining of the subsoil has enabled the necessary precautions against collapse to be taken, and very little serious trouble has been caused. It has also been proved that it is now feasible to have cellars for the buildings.

The speaker believes that these good results are due partly to the good construction of the sewers, partly to the fact that the surface is mostly impervious—owing to the buildings and good pavements and the quick removal of rain water by a special drainage system—and partly to the watchfulness of the authorities.

To build a sewer system in a material that was practically "muck" was certainly an interesting proposition. Of great importance as preliminaries, in the speaker's opinion, were a physical analysis of the soil, a minimum length of open trench, and rapid but very careful construction.

The speaker admits what Mr. Coleman says, relative to the difference in the soils described, but the author's remarks apparently refer also to the Areas 35, 36, and 37 indicated on the map, and these immediately adjoin the City of New Orleans. The soils, therefore, are probably not very different. The speaker wishes to add that, as it is impracticable with ordinary sewer construction to have the sewers

absolutely water-tight, the ground under the city would eventually and gradually drain itself. It was concluded that the shrinkage, so far as it would go, would have to be very slow, and could and should be observed and suitable provisions made to prevent any serious disturbances. Mr. Hering.

J. F. COLEMAN,* M. AM. SOC. C. E.—The speaker has been quite familiar with the author's investigations throughout the entire period of the operations in these peat soils, and has been kept informed of all the work which he has done, and with what the governmental department expects to do. The speaker fully agrees with the remarks of Mr. Morgan to the effect that the information to be gathered by this series of investigations and measurements will be of inestimable value to those who have to deal with drainage and reclamation projects on these peat and muck lands. Mr. Coleman.

The speaker has listened with considerable interest to the remarks of Mr. Hering on the subject; but feels quite sure that Mr. Okey's paper does not intend to deal with the particular kind of soil which Mr. Hering and his associates encountered in connection with the sewerage and drainage of the City of New Orleans.

The judgment which they expressed to the City authorities has been amply sustained since that time, for the reason that there has been no appreciable subsidence of buildings in New Orleans, due to the subsidence of the soil on which they rested; and the only troubles from which buildings have suffered as a result of this drainage has been due to the fact that many of the old ones, which were constructed on spread foundations, had those foundations imposed on grillage of timber; and, in some instances—the number of instances is rapidly multiplying now—the lowering of the line of saturation has caused a decay in the timber grillage, and some foundations are beginning to need treatment as a result.

The soil, however, with which Mr. Okey's paper deals, may be more fairly denominated as peat, in so far as Southern Louisiana is concerned. It is a soil which, under no circumstances, would carry a greater load for spread foundations, for example, than from 100 to 250 lb. per sq. ft., whereas the soil of New Orleans, in the area that has been dealt with in the drainage and sewerage system, will support from 750 to 1 500 lb. per sq. ft.

The soil of New Orleans has been overflowed, in years gone by, by the Mississippi River, and portions of the original muck soil there have been very completely filled over with sand and silt, which would be indicated by the analysis which Mr. Hering mentioned, showing about 50% of sand and quite a considerable quantity of clay in addition; whereas, in the peat or muck soil of Louisiana, as described in Mr. Okey's paper, there is virtually no sand.

* Mobile, Ala.

Mr. Coleman.

On such land, after the water-table has been reduced by the pumping station, and has been thus maintained for a period of months, it takes little or nothing to set fire to that soil during a dry spell; and it will burn down to the water-table. In fact, to fight these fires they excavate ditches surrounding the fire down to the line of saturation; so that they are truly, or almost truly, peat lands. The material may be excavated from them and made into briquettes for fuel.

Mr. Lyle.

WILLIAM T. LYLE,* ASSOC. M. AM. SOC. C. E. (by letter).—An extensive deposit of muck and peat occupied 25% of the area of the tract set apart by the Essex County Park Commission, in Newark, N. J., as Westside Park. This park covers six city blocks, and has an area of 24 acres. The western half of the park is high, and the eastern half low. In the latter part, especially to the southeast, there is a muck and peat deposit having a depth of 35 ft. the development of which was the most important feature of the park work.

The general contract included the following operations. First, the entire site, with the exception of the bog, was stripped of its top soil. This was placed in two large piles with spiral drives. The necessary excavation and embankment was then made to bring the surface to sub-grade. On the site of the bog, the landscape architects, Olmsted Brothers, of Brookline, Mass., planned a large artificial pond involving the removal of muck and peat to the depth of 10 ft. Most of this material was used as filling on the prepared upland sub-grades and in the construction of a border-mound along Fourteenth Street. The remainder was hauled away to Branch Brook Park for use as a fertilizing top soil. To maintain a shore on the eastern side of the pond, the plans of the chief engineer, Howard J. Cole, M. Am. Soc. C. E., called for a timber bulkhead of piles, driven close, with every fifth pile battered to act as a brace. In front of the bulkhead, that is, on the pond side, good upland sub-soil was deposited. After settlement, this fill was trued up to the finished surfaces. The muck and peat were removed from the pond by tram, by wagon, by derrick, and by cableway.

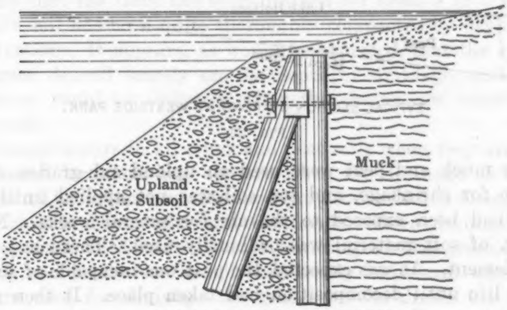
The other main contract items were sewers, water pipes, paths, and retaining walls.

The upper layer of material removed from the lake was black, and is defined as "muck" by Mr. Okey. Its depth was from 4 to 5 ft. It was composed mainly of vegetable matter, but contained a little mineral matter also. The surface of the deposit, before the work was begun, was sufficiently firm to support a loaded wagon. After being dried in an oven, the muck could be burned like coal. Below it there was a brown colored peat, which, in its natural condition, weighed but little more than water. In it could be seen the imprint of the

* Easton, Pa.

leaves of prehistoric trees, and it contained many well-preserved roots or "snags". The peat was found to be very tenacious of its water, even when piled on the shore, but, when dried in an oven, proved to be good fuel. Mr. Lyle.

To construct the bulkhead, 23 000 lin. ft. of piles were driven. In the preliminary investigation which led up to the design, soundings were made along the proposed shore line and at various other places. This work was done by one man. The method was as follows: A hole, 1 ft. or more deep, was first dug in the crust, and a 5-ft. length of gas pipe was driven down with a maul. To it was coupled another length, which was driven in the same manner, and so on until hard bottom was reached. The pipe was raised with a chain and a lever by the man who did the driving. It contained a core of the material through which the sounding rod had passed, with a short core of white sand at the bottom. The same pipe was used repeatedly.



TIMBER BULKHEAD AND SUBSOIL FILLING TO FORM ARTIFICIAL SHORE.

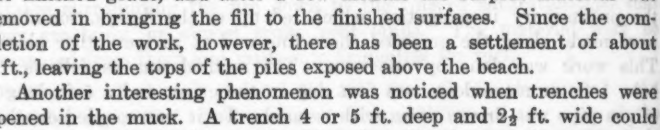
FIG. 20.

During the construction, which took place in 1899 and 1900, and also after the completion of the work, the following interesting observations were made.

The water level shown on the plan, which was set at 196.0, for reasons of economy, has not been maintained, its average value being about 3 ft. lower. As a consequence, the soil to the east and south of the shore line has drained into the lake. Back of the bulkhead, at a place where the depth to the hard sub-stratum is from 30 to 35 ft., the settlement since the completion of the work is about 0.75 ft. In terms of the distance down to the water-table, this settlement is about 15 per cent. There had been a probable equal settlement during construction, thus making the total shrinkage about 30 per cent. In making the fill in front of the timber bulkhead, the engineers were reasonably sure that equilibrium would be attained before the deposited

Mr. Lyle. material reached solid bottom. This was found to be the case. The upland soil dumped over the bulkhead displaced an equal volume of peat which had to be removed by cableway. In order to guarantee a more or less stable equilibrium, the fill was carried about 3 ft. above the finished grade, and after a few months the surplus material was removed in bringing the fill to the finished surfaces. Since the completion of the work, however, there has been a settlement of about 2 ft., leaving the tops of the piles exposed above the beach.

Another interesting phenomenon was noticed when trenches were opened in the muck. A trench 4 or 5 ft. deep and 2½ ft. wide could easily be excavated without sheathing, but, if left open for more than 12 hours, would gradually close, the sides remaining as vertical surfaces. Ditches 2½ ft. wide in the afternoon were sometimes found to be only 6 in. wide the next morning.



TYPICAL SECTION OF POND AT WESTSIDE PARK.

FIG. 21.

Where muck and peat were used on upland sub-grades, allowance was made for shrinkage, and top soil was not applied until the wet material had been exposed to the air for several weeks. Not more than 2 ft. of soft material was ordinarily used. There has been but little settlement. As was expected, the peat was incapable of supporting vegetable life until decomposition had taken place. It then gave rise to luxuriant growths.

Mr. Kimball. WILLIAM H. KIMBALL,* M. AM. SOC. C. E. (by letter).—This paper deserves careful study by those engineers who have to deal with the reclamation of peat and muck soils, and they will follow with interest the continuation of the observations.

The paper and its discussion show that engineers are justified in examining critically some of the previously published conclusions regarding the probable settlement of muck soils after drainage.

The writer is somewhat familiar with Florida drainage work, as he has been in charge of the Vero Drainage District since 1913, is familiar with the investigations made there under Mr. Okey's direction, and believes that the statements show the true conditions for that area. The muck of the Vero District, however, is of shallow depth, compared with other Florida projects, and is not as representative of what may be anticipated as are the investigations in the Everglades.

* Davenport, Iowa.

The paper and its discussion have made evident the great variation in the character of soils referred to as muck and peat, and emphasize the importance of a careful soil analysis in each case. Mr. Kimball.

In the writer's opinion, Mr. Okey's paper is most timely and valuable, and the serious results that may arise from failure to anticipate the subsidence of muck and peat soils after drainage and cultivation are forcibly presented.

S. H. McCrory,* M. Am. Soc. C. E. (by letter).—Since the paper was written by Mr. Okey a number of the profiles have been run again. The data obtained show that subsidence has occurred in each district where the profiles have been repeated. In some districts it has been small, in others quite large. In the Little Woods Drainage District in Orleans Parish, Louisiana, the water was held low in the canals and the soil was well drained; however, the new profile shows very little additional subsidence. Undoubtedly this is due to the fact that the tract has been drained for from 5 to 7 years, and the surface soil has been quite thoroughly compacted by drying, decay, and cultivation. It appears, as would be expected, that the subsidence and its rate depend largely on the depth of drainage, and that the rate is most rapid immediately after the drains are constructed or are deepened. Mr. McCrory.

Subsidence occurs in all muck and peat soils when they are drained, and this must be taken into account when the drainage of such soils is planned. The writer has examined projects in North Carolina, New York, Michigan, Minnesota, and California, where drainage systems have been made ineffective by this lowering of the ground surface. The muck and peat areas usually are nearly level, and when gravity outlets are used for drainage the grades of the drains are very flat. Under such circumstances, lowering the surface only a few feet will decrease seriously the cross-section and the capacity of the ditch, thereby requiring that the drain be deepened or, in some cases, that pumping plants be put in. If the layer of muck or peat is not deep and it is underlain by a good soil, subsidence usually will not be a serious matter; but when this top soil is of considerable depth, or is underlain by hard rock or sand, the construction of a drain that will be permanently satisfactory may be impossible.

The information that the writer has been able to obtain in regard to subsidence of muck and peat soils in the Northern States is rather meager, but apparently the percentage is about the same as in the Southern States, though the rate is somewhat slower.

Charles W. Okey,† Assoc. M. Am. Soc. C. E. (by letter).—The discussion contributed by Mr. Morgan furnishes further valuable information in regard to the characteristics of muck and peat soils, and Mr. Okey.

* Washington, D. C.
† Nashville, Tenn.

Mr. Okey. makes clear the fact that there is a great difference in them, which, of course, is due to the circumstances under which they were formed. Although there are variations, due to the nature of the vegetable life which furnished the vegetable content of the soil, the principal factor to be considered is whether or not the waters near which, or in which, the muck or peat is formed carry silt from other locations into or over those areas. Muck and peat soils are formed in poorly drained areas lower than the surrounding land, and therefore receive the drainage from the higher areas. In relatively small areas of such soils, and particularly when the surrounding slopes are pronounced, there is a heavy percentage of silt. On the other hand, in large areas, such as border the coast of the Atlantic States, as exist in Minnesota, Florida, and Southern Louisiana, the mineral content is low, except for overflows of the larger silt-bearing streams. The soils of Southern Louisiana have a relatively high mineral content because the Mississippi River, with its enormous burden of silt, has actually formed a good portion of the State of Louisiana, and the river has been able to furnish enough flood waters to reach practically every portion of the alluvial region. On the other hand, the Everglades of Florida have been formed in clear waters, and, except for a small strip along the southern shore of Lake Okeechobee, have received no alluvial deposit.

Although the percentage of ash was not determined for the soils examined, samples in sealed containers were sent to the Office of Public Roads and Rural Engineering by the writer when the investigations were made, and, if these samples have been preserved, the determination could yet be made.

Mr. Randolph makes this statement:

"It would appear * * * that the subsidence of saw grass muck, * * * takes places somewhat uniformly over the entire depth of muck, and that it is not confined to that portion of the muck which stands above the water-table."

In the sentence following he also states:

"It also appears that the depth of subsidence due to this cause is dependent on the depth to which the water-table has been lowered, * * *."

On their face, these two statements appear to be contradictory, but perhaps Mr. Randolph's intention is not quite clear. The writer agrees with the second statement, but thinks that the portion of the muck which is permanently saturated does not show any appreciable change.

In securing the samples for determining their weight, the writer used sheet-iron cylinders, 4 in. in diameter, 4 in. in length, and open at both ends. The cutting end was sharpened, and the sample was

secured by forcing the cylinder vertically into the soil. Samples were taken from the top downward, in such a way as to get a continuous column. As water was usually present either on or in the soil, it was not practicable to go very deep. However, as shown by the profiles, in most cases in Southern Louisiana it was possible to get through the top soil to the underlying alluvium. Especially on soils which had never been cultivated, the cutting of the first sample was somewhat difficult, due to the presence of tough roots. As a result a certain amount of compacting occurred, and the 4-in. sample represented a slightly greater depth, as the cutter was always forced into the earth until the material was level with the top. Even in the subsequent samples there was a small amount of compacting, due to the friction of the material entering the cutter. Consequently the sample in the cutter actually represented a slightly greater volume of the material in its original state. Therefore the calculated weight per cubic foot would be slightly in excess of the actual. It is considered likely that the error is greatest on the lightest and softest materials, as they were compacted most when samples were taken. Mr. Okey.

Mr. Hering's discussion is applicable to such soils as are found in New Orleans and in rather close proximity to the Mississippi River. These are almost entirely alluvium, and, in comparison with the muck and peat lands, contain very little vegetable matter. Although the areas numbered 35, 36, and 37 on the map are not far from New Orleans, the soil conditions are entirely different, at least so far as the portion of the city near the river is concerned.

The material described by Mr. Lyle is evidently well-decayed muck—at least the upper portion. It would be very interesting to have observations repeated on this piece of work in the future.

The discussion contributed by Mr. McCrory is exceedingly interesting, as it shows the additional settlement in the districts, examined by the writer 2 years before. The measurements were made by other observers, and the writer did not know of the results. It is to be hoped that the work will be continued. After a period of about 5 years, or in 1920, the writer believes that it would be of value to have samples of the soil taken under the same conditions as in 1915.

The writer is very grateful for the favorable discussion contributed by the several members. Mr. Morgan, Mr. Kimball, and Mr. McCrory have each put special stress on the importance of anticipating the subsidence of muck and peat soils in the design of drainage improvements. The writer agrees with these gentlemen in every point they have made. It is now a certainty that, unless provision is made for such subsidence, there is trouble in store for the owners of lands such as these, which are drained by channels adequate only if the surface maintains its original elevation.

Mr. Okey. The writer regrets that engineers who are responsible for the design of drainage improvements for some of the largest areas of muck and peat lands in the world have not been contributors to the discussion. At the time the paper was opened for discussion the writer furnished a list of names of members who were directly interested in such work, and these members were requested by the Secretary of the Society to discuss the paper or to offer such material as they had available on the subsidence of muck and peat soils. Discussion from those engineers, responsible for the drainage of such great areas of lands, would be of the highest interest. The writer feels that the membership would then receive the benefit of the practical application of the facts, in regard to the subsidence of muck and peat soils, to the concrete problems presented in the design of drainage improvements. It would be exceedingly interesting and instructive to know what provision is being made for subsidence in the design of these great works, in which millions of dollars will ultimately be expended, and on which thousands of farmers will depend for adequate drainage. It is to be hoped that such discussion will be forthcoming.

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Paper No. 1411

MODERN PRACTICE IN WOOD STAVE PIPE DESIGN AND SUGGESTIONS FOR STANDARD SPECIFICATIONS*

By J. F. Partridge, Jun. Am. Soc. C. E.

With Discussion by Messrs. Hermann von Schrenk, Frank F. Bell, D. C. Henny, Henry P. Rust, F. M. Robbins, William J. Boucher, J. C. Ralston, O. P. M. Goss, W. H. R. Nimmo, E. A. Moritz, and A. N. Miller.

Synopsis.

The object of this paper is to give engineers an idea of the difference between the various grades of wood pipes; to set forth a standard set of specifications for the assistance of engineers who have had no opportunity to become versed in their design; to safeguard those who contemplate building such pipe; and, further, to remove doubt from the minds of those who view wood pipe as one of the vagaries of engineering practice and a medium to be resorted to only in temporary and cheap work. If it can be shown that, to secure good results, the great difference in the quality of the materials used should be completely borne in mind, and if engineers can be led along a correct and standard course in the design and in the selection of these materials, this paper will have accomplished its object. To this end, specifications involving the latest and most approved practices are given in the Appendix.

* Presented at the meeting of May 16th, 1917.

Note.—The author of this paper is now in the Military Service of the United States, and is prevented by his duties from contributing a closing discussion.

The elements causing success or failure in wood stave pipe are taken up step by step, as follows:

1.—Kinds of wood used,

2.—Grade of lumber used,

3.—Method of curing lumber,

4.—Method of treating lumber,

5.—Location of pipe when built,

6.—Size and spacing of bands,

7.—Methods used in erection, and quality of workmanship.

The foregoing headings are discussed as applied to the two types now in use, namely, continuous-stave pipe and machine-banded pipe, and a plea is made for the adoption of uniform specifications, dividing each type into Classes *A*, *B*, and *C*.

The Appendix contains the specifications, for the two types and three classes of pipes, which are proposed as a basis for adoption by engineers.

In 1898, the late Arthur L. Adams, M. Am. Soc. C. E., presented to this Society a paper* entitled, "Stave Pipe—Its Economic Design and the Economy of Its Use." This was the first important presentation of wood stave pipe design, and it brought forth a great deal of discussion. Mr. Adams prophesied the value that wood pipe would attain in hydraulic engineering, and was the first to give it its proper place with reference to cost, life, and capacity. He placed it first in economy of construction, first in carrying capacity, and second only to cast iron in length of life. These statements were rather startling to most engineers at that time, as they considered his deductions based on insufficient data. In 1906 Mr. Adams presented another paper† discussing the famous pipe of the Astoria City Water-Works. The discussion on that paper gave to engineers for the first time an idea of what could be expected of wood stave pipe. However, it was thought that a few more years must elapse before the real economy of such pipe could be determined. This was conservatism, for which engineers are noted, and proved to be a wise policy. Mr. Adams foresaw the value of wood pipe, but experience showed that all such pipe could

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 27.

† *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 65.

not be placed in one class. Development with years made it evident that careful design and selection of material were necessary, without which complete pipe failures would result. The indiscriminate use of various woods, which time showed to be unfit for good pipe construction, proved that it was wise to make haste slowly.

Though Mr. Adams foresaw the ultimate success to be reached by wood pipe, he did not foresee the rapid strides, and consequently the hasty, unscientific, and indiscriminate use of all kinds of materials; and these proved totally unsuited for pipe design, and threatened to turn success into failure.

In various engineering journals, in recent years, much space has been devoted to this subject, and the articles written have thrown considerable light on what had been but a hazy understanding in the minds of most engineers. The advantages and disadvantages of wood stave pipe have been discussed, but these discussions have only led to greater confusion and doubt as to the best way to use it. Consequently, a most valuable asset to hydraulic work has been shoved into the background and considered as a type to be used only in special cases.

In all the discussions it is noticeable that there are no references to cases where wood stave pipe has been constructed or operated successfully, or unsuccessfully, for a number of years. Actual cases where it has been in service long enough to give an idea of its durability are wanted by engineers. With this information, they can give some assurance that, if such pipe is used, it is the proper construction.

In most of the articles referred to, statements like the following are the general rule: "Experience shows that staves must be completely and continuously saturated, and that intermittent or partial saturation leads to decay;" "a stave pipe is extremely short-lived, even when made of the very best selected wood, under partial saturation, especially in warm, humid atmospheres." Warm, humid atmospheres are often encountered in localities in which engineers may have wood pipe under advisement, and conditions of partial saturation or drying out of the pipe during a portion of each year are often unavoidable. Such statements, therefore, are misleading, as the general impression is that favorable conditions for wood pipe are few, and, under all other than favorable conditions, it will probably be short-lived. This is not

true of well-designed wood pipe, and especially does it give a false idea of its usefulness when made from the proper materials.

A few cases that contradict the general impression regarding such pipe under conditions of intermittent flow and partial saturation are well illustrated by the following:

1.—Supply line of the Utah Lake, Land, Water, and Power Company, at Mt. Nebo, Utah. A half pipe or flume and a 48-in. pipe built in May, 1893. Intermittent and partial flow during a few months of the year. At maximum flow under 70 ft. head. Pipe entirely above ground, and for part of the way bracketed against a rock cliff, with exposure to the south and the full heat of the sun. Inspection in October, 1914, showed pipe to be without decay. Clear redwood staves.

2.—Discharge line from sugar factory of Los Alamitos Land Company, Los Alamitos, Cal. Mr. H. C. Lawrence, Chief Engineer. Built in 1902. Used only for 4 months of the year. Discharge for refuse from factory; operating under no pressure. Mr. Lawrence states that the line is in very good condition; a few bands show corrosion, and a good many have been replaced. He estimates that the pipe will have a life of 50 years. Clear redwood staves.

3.—Sewer for Palo Alto, Cal. Built in 1898. Continuous flow, from one-half to three-quarters full. Pipe extends across salt marshes bordering San Francisco Bay; exposed at low and covered at high tide. Portions of line buried completely, half buried part of the way, and remainder exposed on the surface. Edwin Duryea, M. Am. Soc. C. E., in his discussion on Mr. Adams' paper of 1906, stated that this pipe showed not the slightest decay, though the bands had corroded badly. Air exposure, contact with the humus in the soil, saline soil, and partial saturation only, seem to have had no bad effect on the pipe, which is to-day in perfect condition and operating continually. A portion of the line buried in sandy soil had to be repaired a few years ago, some of the top staves requiring replacing, due probably to the sandy soil drawing out what little saturation these staves received. Clear redwood staves.

4.—Water supply pipe line for San Diego and Coronado, Cal. Built in July, 1900. 13 550 ft. of 40-in. and 26 300 ft. of 36-in. carry water from Otay Dam to these cities. Maximum pressure, 295 ft.; minimum, 150 ft. Pipe buried for entire distance in alkali flats, but

above ground where several deep ravines are crossed on trestles. Examination by writer in 1916 showed staves to be in perfect condition. Mr. O. D. Fees, Superintendent in charge of line during construction, accompanied the writer, and stated that the wood was considerably harder than when first put in. It is quite possible that solubles carried in the water have entered and have been deposited in the pores of the wood. This is redwood pipe.

5.—Leaching tanks in the plant of the Krieg Tannery, San Francisco. Built in 1859. A number of tanks above ground leaching into those below ground. Removed in January, 1914, found to be in perfect condition, replaced, and now in service. Redwood staves used.

6.—A redwood flume built in 1888 for the Cuyamaca Water Company, San Diego, Cal., has much of its original lumber in place to-day.

7.—Two 32-in. inverted siphons, one 20 and one 15 years old, in the line of the Yakima Valley Canal Company, North Yakima, Wash., which operates during the summer only, showed tapered ends of staves in upper part decayed when the pipes were torn out to be replaced by a 48-in. line to increase the capacity. Redwood staves were used in all three pipes.

The writer's criticisms of the articles relating to wood pipe will have to be modified because of the recent publication* of a paper by D. C. Henny, M. Am. Soc. C. E., Consulting Engineer for the United States Reclamation Service. In this paper Mr. Henny made the first attempt to segregate the various types and grades of wood pipe. He presents valuable data as to what can be expected of the average wood pipe, made from various materials, and operating under various conditions.

The discussion, however, should be carried still further, and the facts regarding manufacture and design that will give the pipe the expected life should be investigated and presented, so that engineers can determine intelligently the type that will best fulfill their conditions.

Wood pipe is too often classed as a whole, irrespective of the material from which it is made, no attention being given to the fact that there is as much difference between the various makes as between cast-iron and steel pipe, in fact, more. It is quite possible to make a

* In the *Reclamation Record*.

run of steel or iron with identical quantities of impurities, thus obtaining practically uniform products. Wood, on the other hand, is the most variable material known to the structural engineer, and is acknowledged as such. Yet, in discussing wood pipe, no distinction is made as to quality, which depends on the kind of lumber used in the staves.

On work of any magnitude, where prominent engineers are consulted, conduits are generally chosen after deep study, and the results usually prove worth the expense of expert investigation. There are countless conduits, however, throughout the United States, where cheapness has superseded economy, and the resulting failures have shaken the faith in the type. Wood pipe has suffered the most. The many conduits with staves of inferior wood and poor manufacture have made engineers and others skeptical of this type of construction.

Wood pipe of the stave variety—and this is the only type considered to-day—was primarily a product of the West, although first invented and built in the New England States. The high freight rates on steel (which had to come from the East) made steel pipes very expensive, and the large quantities of timber available in the West made the use of wood an economical necessity. To build stave pipe, timber must first be available, and then the proper machinery to mill the staves, otherwise, it would not pay to use this type, except in those few cases where large projects warrant the cost of erecting machinery to mill the staves. Companies with timber holdings and mills of their own were naturally in the best position to manufacture such pipe. As a consequence, this business drifted into their hands, and they undertook it merely to sell lumber, only a few companies being formed to construct such conduits.

To-day it is possible for individuals to obtain materials and bid on wood stave pipe contracts, and such work is often undertaken at absurdly low figures in competition with experienced companies, which, knowing their business, are unable to secure the work except at a heavy loss. The successful bidder does the work to the best of his ability, but, as the building of continuous-stave pipe requires years of experience, he loses money, the pipe manufacturers are unable to keep in the business, and the purchaser secures a pipe that never proves a success. It is the duty of the engineer to protect his employer

against such conditions, and to do so he must be fortified with good specifications and must enforce compliance with them.

The elements causing success or failure in wood stave pipe include:

1.—Kinds of wood used,
2.—Grade of lumber used,
3.—Method of curing lumber,
4.—Method of treating lumber,
5.—Location of pipe when built,
6.—Size and spacing of bands,
7.—Methods used in erection, and quality of workmanship.

Redwood, fir, cypress, and pine are in general use, and make pipes of different characteristics. The pipe is also affected by the sap, pitch, or knots in the staves. The method of curing—kiln or air-drying—also influences the quality. Treating lumber with creosote, or surface painting, also affects the final result. The location determines to a certain extent the type of pipe to be chosen, and the size and spacing of the bands and the methods used in erection make a first-class or a useless pipe out of the materials available. If an engineer knows only the general methods of construction, and not the fine points, he cannot build a wood stave pipe line as well as a company which has had years of experience, and is likely to have trouble.

A discussion of the merits or demerits of such construction is misleading unless based on a clear specification. It is known, of course, that wood pipe kept constantly saturated will last indefinitely, but, as such cases are not always found, one must consider what will happen under other conditions. Fir and pine are pitchy woods, and it is impossible to obtain commercial run lumber without sap, pitch, pitch seams, pitch pockets, and knots. Under conditions of partial saturation, this lumber will not last, and, even with saturation, the pitch and sap will be the cause of deterioration. Most failures are attributable to this fact. There are conditions under which fir or pine will have a long life and give perfect satisfaction. For instance, erected on cradles, allowing the air to circulate freely around it, pipe will give satisfaction if the climate is dry, so that mosses, etc., caused by dampness, do not accumulate on the exterior. Pipe under heavy pressure in compact soil will last indefinitely. Mr. Henny has given the following tabulation:

Wood.	Condition.	Years.
Fir.......	Uncoated, buried in tight soil.....	20
"	" " " loose "	4–7
"	" in air.................	12–20
Redwood..	" buried in tight soil, loam, sand, and gravel........	(More than 25)
Fir.......	Well-coated, buried in tight soil...	25
"	" " " " loose " ...	15–20

Cypress makes a most excellent and durable pipe, and is the only competitor of redwood with reference to length of life and endurance under alternately wet and dry conditions. If cypress is selected so as to eliminate sap, it probably is as long-lived as redwood; at least, it is near enough to avoid discussion. The disadvantages of cypress are:

First, the quantity of standing timber is extremely limited, and it is estimated by conservative lumbermen that all the commercially available cypress will be cut during the next 10 years. Thus, those who have cypress are constantly raising their prices to correspond with the advancing rise of stumpage.

The second disadvantage is in the wood itself. It grows in swamp land, and the butts of the trees are usually under water. A cypress tree is the product of four or five small trees growing together. The result is that the sap does not come as it does in redwood, entirely around the circumference of the tree, extending inward only 2 or 3 in., but it occurs throughout the clear part of the log, in streaks or strips. It is common to see clear cypress with yellow sap streaks running through the center at intervals of about 4 in. This has brought about the peculiar condition, that cypress has a grade higher than clear, and (the writer believes) is the only lumber which is thus graded. "Tank" grade is the highest in cypress, and contains knots but eliminates sap. Cypress knots are smaller and harder than those of redwood, and are not as detrimental.

It is extremely difficult to get cypress for pipes, because of the sap, and, where sap is eliminated, the price of the wood is so high that it cannot compete. Cypress pipes are rare.

Redwood is the best known material for wood pipe, and its longevity is excelled only by cast iron. The acid or other peculiar constituent of this wood acts as a preservative or micro-organism destroyer, and protects and preserves it.

The cases of redwood pipe already cited illustrate its adaptability, whether laid on the surface of the ground, partly or completely buried, or run through salt marshes or tropical swamps in direct contact with the soil humus. Direct exposure to the rays of the desert sun, and alternate wetting and drying when the pipe is used intermittently in irrigation systems, do not lessen its efficiency.

A thorough study of the conditions under which a proposed pipe will operate and an investigation of the materials best suited to withstand these conditions, should be made before specifications are written. These important points should be kept in mind in order to insure specifications that will cover the conditions closely.

Engineers should first decide the nature of the conduit they intend to build; that is, whether it is to be a permanent structure or is to last only 5 or 10 years, after which time it is to be abandoned or replaced by a conduit of increased carrying capacity. Then the nature of the local conditions relative to the pipe line should be ascertained, including climate, humidity, temperature, extreme and average pressure, nature of soil, probability of the pipe being buried or laid on the surface, and other details; and then specifications for the materials can be written.

There is great necessity for uniformity in drafting specifications, and for an understanding of the requirements for securing pipe that will fulfill the needs of the proposed work. At present practically every piece of work is covered by specifications embodying different fundamentals. This is most noticeable in a comparison of specifications for various projects of the Reclamation Service. On some of these a distinction is made between redwood and fir pipe, bids being asked for coated fir and uncoated redwood, though in other projects no such distinction is made, these woods being placed on an equal basis. This question of the coating offers the largest field for disagreement, most engineers being of the opinion that both redwood and fir pipe should be painted in order to obtain good results; on the other hand, many who have had experience with uncoated redwood claim that painting it is unnecessary.

The thickness of the staves is another point of difference, and has been settled theoretically and practically with widely varying results. There is some difference of opinion as to the spacing of bands, depending on the assumed factor of safety, which factor in turn is determined by

the greater or less conservatism of the engineer. In the specifications for the staves is found the greatest divergence, and without justification. It is not evident why pitch and knots should be allowed in fir and not in redwood staves. Another objectionable feature in some specifications is the provision for rigid supervision of the bands, though adequate stress is not laid on the requirements for the shoes, which, after all, must be capable of developing the full strength of the band. The tongues are seemingly the smallest item of continuous-stave pipe construction, but, nevertheless, are by no means the least important. In machine-banded pipe there is absolutely no basis for the present-day so-called specifications.

The result is that some pipe lines are well, and some poorly, designed, the latter very often being the least economical. The purchaser, paying for what he believes to be the best type obtainable, secures a piece of work which proves a failure. These failures hurt the owner and undermine the faith in such construction. A great many engineers have little or no idea of how to design a wood stave pipe, and when it is necessary for them to draw up specifications they seek everywhere for information and acquire and compile a heterogeneous mass of data which are mostly useless.

For the assistance of engineers who have had no opportunity to become versed in wood stave pipe design, and to safeguard those who contemplate building such pipe, specifications should be standardized. The Appendix contains the specifications suggested as the foundation for a standard.

Sap and pitch in the staves mean a short life for the pipe, as deterioration will start first in sap wood, pitch seams, or pitch pockets, and spread rapidly to the clear wood. Pine and fir cannot be secured commercially without these defects, and, therefore, are fundamentally inferior to redwood, in which absolutely clear staves can be easily obtained. At repeated intervals, heavy applications of some protective paint with disinfectant qualities will allay the danger of deterioration in fir and pine, but proof that their ultimate life will equal that of redwood has not yet been obtained.

The thickness of staves should next be considered. Of course, the thicker the stave the better the pipe, but this has economical limits. A thickness of ⅛ or ¼ in., more or less, should not be the subject of controversy between engineers. The best criterion of the required thick-

ness of staves is actual experience. Throughout one section of pipe there will be staves of entirely different characteristics, including grain, resistance to percolation, and ease of penetration. Slash grain, vertical grain, quarter-sawed staves, heart wood, etc., all have their influence on the thickness required, and, with such a great difference in the characteristics of each stave, a small difference in thickness does not affect the quality of the finished pipe.

In designing staves there is more than the thickness to be considered. Economy is the other essential feature. In common practice, stock sizes of lumber are chosen which will give the most economical number of staves to the linear foot of pipe. For instance, for a 36-in. pipe, 2 by 6-in. lumber is chosen. A maximum thickness and width of stave is obtained from lumber of this size, and a certain number of feet, board measure, is obtained in the cross-section of the pipe. If 2 by 4-in. stock is chosen, in comparison with 2 by 6-in., the result may be a saving in the board measure, but the cost of erection of the pipe will be increased considerably on account of the greater number of staves to be handled. If 2 by 8-in. is used, a saving in erection is obtained, but the greater waste in lumber offsets the saving in erection. The result is that 2 by 6-in. is the most economical size. The maximum that can be obtained from this stock piece is the thickness to be specified. It should be remembered, however, that this maximum will be less than that obtained by laying out the stave on paper, showing it cut from 2 by 6-in. stock of exact dimensions. The stock as it comes from the mills, dry and ready for stave manufacture, will probably measure not more than $1\frac{7}{8}$ by $5\frac{3}{4}$-in. Allowing enough for milling, the thickness of the stave will be reduced. Common practice and experience in stave milling should always be considered. If greater thicknesses are wanted, a higher price may be expected, as uneconomical sizes of lumber must be used, or a higher price must be charged to cover the selection of wider and thicker stock.

The stave has to resist the percolation and the penetration of the water. It should be sufficiently thick to prevent excessive percolation, and, at the same time, there should be perfect penetration. It is difficult to determine this thickness. If the staves have rings showing wide, alternate spaces of hard winter wood and soft summer wood, there will be great danger of excessive percolation, the water finding its way out through the soft wood between the hard rings. If the wood is very

hard, there will be great difficulty in the stave receiving complete saturation, due to the absence of capillary action. A soft wood will take up water like a blotter, but a close-grained wood will effectively resist percolation. This is very important in determining the lumber to be used.

Fir and pine, being hard woods compared with redwood, and being coarse-grained, having wide rings of hard and soft wood, enter the classification of woods giving excessive percolation, with slow and incomplete penetration. This is caused by the water passing rapidly through the soft summer wood, appearing in drops on the outer surface of the pipe, and of penetrating but slowly, and often through only a fraction of a stave, along the hard winter rings. The result is a stave showing percolation and incomplete penetration at alternate points throughout its cross-section.

Redwood is very soft and cellular, and pipe made from clear stock will be free from percolation and will receive complete saturation, even under very light pressure.

There is such a great variation in the quality, grain, and degree of hardness of even the same kinds of woods, that it is impossible to secure a pipe in which the penetration and percolation will be of the same degree in every stave. In the same section of pipe, soft staves with good penetration and no percolation will be found adjacent to hard staves with poor penetration and excessive percolation. It is obvious, therefore, that a refinement of stave specifications to the point of $\frac{1}{8}$ or $\frac{1}{4}$ in. more than a practical working thickness is entirely unnecessary.

In the Appendix practical working thicknesses for staves are given, and it is recommended that engineers give them their attention when drawing up specifications. By a practical working thickness is meant that which can be secured from the stock sizes of lumber making up the most economical pipe.

The best selection of a stave, therefore, is the result of experience with those thicknesses which give maximum penetration and minimum danger of percolation.

It should be remembered that fir or pine staves require greater thickness than redwood, in order to resist excessive percolation, and the result is not altogether beneficial, as the penetration is less likely to be complete.

The difference in required thicknesses of fir or pine and of redwood staves applies more particularly to machine-banded pipe, because the thicknesses for continuous-stave pipe are determined primarily by construction conditions, rather than by reason of penetration and percolation.

One objection to wood pipe is the danger that it may dry out if the water is drawn off. To avoid this, the staves should be thoroughly dry, so that, when properly erected and cinched tight, there will be no leakage. The pipe should be tight and stay tight. If wet staves are used, no swelling can be relied on for making a tight line, and the requisite pressure between the staves to prevent the passage of the water must be the result of cinching the bands. This is practically impossible. The lumber, therefore, should be perfectly dry before being used. It should be dried by the natural or air-drying process, not by the forced or kiln-drying process. By air-drying only is perfect, sound, strong lumber obtained. Kiln-drying makes brittle and lifeless lumber. Air-drying requires time, and, as lumber should be seasoned for at least a year for the best construction, a large stock of it should be available at all times.

The old method of drying lumber (in Maine and Michigan) was by the use of live-steam kilns. These are still used in the Northwest for drying fir and pine, as such treatment is necessary in curing pitchy and sappy woods. The kilns are large rooms, along the floors of which there are perforated steam pipes into which live steam is turned. The lumber placed in such a kiln is literally cooked. Redwood when first marketed had never been kiln-dried, but when the demand became too great for the supply of air-dried lumber that could be kept on hand, kiln-drying was adopted. Redwood treated by this process was flinty, and could be broken into splinters over the knee. Such methods of drying are now being used by some of the redwood mills, but lumber thus treated should never be allowed in pipe construction. The later method of kiln-drying is by indirect heating with steam. Steam is introduced into pipes laid on the floor of the kiln, and air with a certain humidity is admitted into the kiln at a given temperature, is heated by passing up around the steam pipes, and, rising through the lumber, removes the moisture. The air then passes down the compartments at the sides of the kiln where the water it contains is condensed, and the cooled air is again brought down to the heating

pipes. A circulation of air is thus effected by which the lumber is dried. This is far superior to the old method. The introduction of green lumber into a kiln and the forced removal of the water causes a forced and sudden hardening and closing of the pores, checking, splitting, and rendering the wood brittle. Air-drying is a natural seasoning, the slower the better, and is brought about by the wind blowing through properly stacked lumber. When securing lumber for pipe staves, there should be a strict investigation into the methods of drying used by the mills. For correct pipe design, only air-dried lumber should be specified.

In regard to the protection of the staves by applications of coatings of paint or disinfectant, little of value can be cited. Many claims are made for the benefits derived from various coatings, but sufficient data are yet lacking for reliable conclusions. It is certain that such protection increases the life of fir, pine, or other woods containing sap and pitch, but its merits on a redwood pipe have not yet been proved. Though uncoated fir and pine, except under conditions of complete and continued saturation, have proved short-lived, similar pipes coated with a mixture of tar and asphaltum have given far better service, and in many cases appear to be in perfect condition. More than this is not known. The oldest lines, on the other hand, made of redwood, have never been coated with any protective coating, and are still in perfect condition.

A coating, to be effective, should be applied diligently and often. At least two coats should be applied primarily, by conscientious workmen or by some pneumatic process. As in all painting, the personal equation of the workman is 75% of the job. A coating of at least $\frac{1}{16}$ in. should be the result of the first painting, and repeated examination should be made of the line, and the pipe painted every year or so.

Steel bands can be obtained to-day from practically all the large steel mills. These bands are manufactured according to standard specifications; if other specifications are used the cost is greatly increased, and very often it is practically impossible to obtain them. Engineers should bear this in mind, as they will save the pipe constructors much trouble and the purchaser much needless expense by using the standard specifications. The band commonly used is of mild, open-hearth steel, having a tensile strength of from 55 000 to 65 000 lb. per sq. in., with a button head at one end and at least 5 in. of cold-

rolled thread on upset ends at the other. The requirements for pipe bands are included in the standard specifications in the Appendix. Specifications often call for pure iron bands, but, as pure iron is made only in very limited quantities, mostly in Norway and Sweden, it is evident that it would be impossible to comply with such specifications.

The size of the band steel and the spacing of the bands on the pipe are, after all, the most important factors affecting the strength of the pipe. Common practice requires the bands to be spaced so that they will have a factor of safety of four against breaking due to tension caused by the water pressure, though some specifications call for a factor of safety of five. The latter requirement adds greatly to the cost without benefiting the pipe. The factor of safety of four gives ample protection against failure under water pressure through rupture of the bands, but the point to be borne in mind is that the pipe may fail on account of the bands sinking into the wood and allowing the longitudinal joints between the staves to open, thus causing leaks. The failure of the pipe in bearing, however, in pipe more than 10 in. in diameter, is prevented if the bands are spaced with a factor of safety of four in the tension formula. The two formulas to be used in spacing the bands on the pipe relate to the tension in the bands and the bearing of the bands and staves. In the bearing formula the value to be determined by experiment is the strength of the wood in bearing. This has generally been taken to be greater for fir than for redwood, but experiment shows that staves in a saturated condition, as would occur in a pipe in place, have practically the same strength in bearing. This approximates a working stress of 800 lb. per sq. in. If the spacing of the bands is checked by both formulas it will be found that all pipes more than 10 in. in diameter will be designed according to the tension formula, and that the bearing will be amply cared for under this condition.

In the design of every pipe line there will be two or three sizes of bands that may be used, with their corresponding spacing, and it is necessary to choose the most economical. The smaller the band the closer the spacing, and the closer the spacing the better the pipe. Economy limits this to a certain extent, as the smaller bands cost more than the larger ones, and erection costs increase with the number of bands handled. A certain maximum spacing should not be exceeded, in good pipe design, and the size of the band may be cut down to hold

this spacing to a minimum, maintain the factor of safety, and still give economical erection. To hold to the maximum spacing with a large band wastes metal, but may be found to be most economical on account of the higher price of small bands. Erection has an important influence on the size of the bands on light-pressure pipe with the maximum allowable spacing, because, if small bands are used, it may be found impossible to cinch them tight enough to prevent seam leaks, and the threads on a great many bands will be stripped before the staves can be drawn together tightly enough to prevent such leakage. Heavy bands are required to draw the staves together, but after they are in place the initial tension in the bands is dissipated, and the only stress is that due to water pressure.

By no means the least important feature is the design of the shoes for cinching the bands tightly in place. These should be stronger in body than the bands, in order to develop the full strength of the latter. There should be a thorough investigation of the shoes, and, unless a special test is made, those that are standard and have been tested repeatedly should be used. It is folly to have bands spaced with a high factor of safety and then use shoes which are weaker in design than the bands they hold together. A case is known where this happened, and approximately 390 tons of steel were absolutely wasted. On some recent municipal work for a large western city a similar case of inferior shoes has lessened the efficiency of the conduit.

The tongues prevent the staves from working out at the butt joints, and are intended to form an effective water seal where the staves butt together. They are generally made of band iron, $1\frac{1}{2}$ in. wide, No. 12 or No. 10 gauge, or $\frac{1}{8}$ in. in thickness, cut $\frac{1}{8}$ in. longer than the width of the stave measured along the slot. This allows $\frac{1}{16}$ in. to project into each adjoining stave, effectually preventing water from working around the joint. When the pipe is first cinched up, prior to rounding out the staves to the true circle, these tongues are set farther than this $\frac{1}{16}$ in. into the adjoining staves. Then, when it is attempted to round out the pipe, that is, hammer the stave from the inside into proper position, the tongues will tear the staves badly, and decay first starts where lumber is bruised or torn. Contact is all that is needed for a perfect water seal, and if the tongues are cut $\frac{1}{16}$ in. longer than the width, that is, cut to allow an initial projection of $\frac{1}{32}$ in. in the adjoining staves, a water-tight joint will be secured with less damage

to the wood during the rounding out of the pipe. There is little likelihood that the tongues will rust out, as air does not reach them readily; however, they are generally coated with an asphaltic base paint.

The rod of the least diameter that can be used on light-pressure pipe, with maximum spacing, can only be determined by experience, and should not exceed the sizes given in Table 1.

TABLE 1.

Diameter of pipe, in inches.	Minimum size of rod, in inches.	Maximum allowable spacing, in inches.
12 to 24	3/8	10
24 to 36	7/16	10
36 to 48	1/2	10
48 to 72	5/8	10
72	5/8	8
72 to 96	3/4	10
96 to 132	3/4	8
132 to 144	3/4	6

When the maximum spacing is used, extra bands should be placed over the butt joints to reinforce the pipe at these points. Many pipe failures have resulted from allowing too wide a spacing as a maximum, and such conduits, under a heavy back-filling, failed by the arch of the pipe collapsing. A maximum spacing of 18 in. has been used, but this does not make a pipe. A spacing of 12 in. has been used with success, but the best results are obtained with a maximum of 10 in.

The coating for the bands and shoes is determined by the conditions and the life to be expected of the pipe. For the bands of fir and pine pipe an asphaltum coating is sufficient, as the bands will outlast the staves. On redwood, contrary to the general impression, the life of the pipe will invariably be the life of the bands. The life of the pipe, therefore—or its economy—depends on the coating first applied to the bands; in after years the pipe should be well inspected and the bands repainted and replaced when necessary. A coating having an asphaltic base is most commonly used, and gives perfect satisfaction. Under normal conditions bands thus coated last from 10 to 15 years, after which time it will probably be necessary to replace them occasionally, although, as a whole, they will outlast steel pipe, as the metal is concentrated in a round band which presents a minimum surface to corrosion. Covering with red lead is recommended for severe conditions, bands having been found as bright under the red lead as when first erected, even after 26 years' service. In Central America coating

with red lead is found to be well suited for protection against the ravages of tropical climatic and soil conditions, and exposure to the salt air during steamer transit. Galvanizing may be used, but, on account of its excessive cost, is not common. In the Hawaiian Islands galvanizing is used exclusively, together with redwood lumber, and such design has been found to give practically the only pipe that will stand up under the conditions there.

No attempt will be made to outline the best methods of constructing wood stave pipe, because there are so many details that require attention and experience that, unless an engineer is familiar with such work, he will do better by securing the services of reliable pipe constructors.

Machine-banded or wire-wound pipe has come into use since Mr. Adams presented his last paper on wood stave pipe, and has found a ready market in the West, on account of its economy. It is factory made, in sizes from 2 to 24 in. inside diameter, and is designed for heads up to 400 ft. The sections are from 8 to 24 ft. long, and have the necessary couplings or collars for connecting them.

Since this type made its appearance, some time ago, there has been practically no improvement, and the old specifications and methods of manufacture are still followed. In spite of its many imperfections in design and manufacture, this type has made a wide field for itself, and is found in every branch of hydraulic work. Unless steps are taken to correct its weaknesses, however, it will rapidly lose favor on account of its numerous failures. These failures have not been altogether the result of poor manufacture, but have been due to an endeavor to reduce the cost. This was done to such an extent that good manufacture and design were impossible.

It is subject to the same criticism as continuous-stave pipe. Conduits of poor design and poor material, or material unsuited for the work, have given the impression that wood pipe as a whole is unsatisfactory and short-lived.

Modern machine-banded pipe is made with heavy staves, generally kiln-dried, banded with galvanized wire, from No. 6 to No. 00 gauge, spaced according to the pressure. The wire is securely fastened to the pipe with pressed-steel clips or staples, or both. Each section is dipped in asphalt and rolled in saw-dust, the asphalt effectually covering the wire and staves, and the adhering saw-dust permitting it to be handled

readily. The sections of light-pressure pipes are joined with inserted or slip-joint connections, being sometimes reinforced with a steel band equipped with a shoe for cinching tight. On high-pressure pipes (generally for more than 100 ft. static head) collars are used. These collars are made in a manner similar to the pipe, the sections being tapered and driven firmly into them. On pipes of large diameter, operating under heavy pressure, the collars have individual bands fitted with shoes for cinching. Riveted steel or cast-iron collars, with or without bells, are also used.

The foregoing describes the pipe generally made by all manufacturers of this type, and represents standard practice. Numerous failures have rendered the recommendation of this type doubtful. If investigations were made into the actual causes of the failures, the reasons would be plain, similar designs would be avoided, and there would be rigid inspection of the manufacture. In outlining the results of the methods of manufacture, it will be well to mention the reason that fir pipe is the butt of the criticism. Fir is the pipe that has failed, the oldest lines having been built not more than 10 years, the greater number being of comparatively recent date. When these pipes were made, fir was chosen as it was the cheaper wood, and there was no criterion as to longevity. The greater number of failures possibly originated at the joints. The outer edges of the staves in the collars, when wood collars were used, decayed rapidly owing to the fact that fir needs saturation for preservation, and saturation was not secured at those places. Cast-iron and steel collars were the remedy, but have not proved successful, owing to their high cost, the increased weight, and the difficulty of making tight connections and plugging leaks. Riveted steel collars can be used to advantage on fir or pine pipe, as they will last as long as the staves, in which the sap wood decays rapidly.

Redwood collars of the individual banded type are used for repairing pipes which have failed at the joints. The decayed fir collar is cut off, and the redwood staves are put in position and cinched tight. This method requires no further attention, and can be applied successfully at any joint where there is decay. The wire is cut away and securely stapled, and the redwood collar is put in position.

The use of the inserted or slip-joint pipe is not to be recommended. Such a connection weakens the end of every section, because nearly one-half of the shell of the pipe is cut away to make the joint. A

reinforcing rod is often used to draw the joint tight, but if the male and female tenons are eccentric, leakage cannot be avoided. There is also great danger of injury to the pipe by handling, before and during shipment, as well as in laying; the weakened ends are not reinforced, and often split off with rough handling. The collar connection is to be preferred, as it will insure a better and stronger pipe, and the greater length of life will warrant the increased cost.

Most of the serious failures have occurred when the water has been drawn out of the conduit for any length of time; this has caused the staves to dry out and the pipe to fall to pieces. Failures by the galvanized wire breaking, mainly at the twist splices, are serious, as the entire section of pipe on which such a splice occurs must be removed.

As a remedy for failure by decay, machine-banded pipes are now painted or dipped in a protective coating, but the same conditions exist here as with the continuous-stave pipe, and pitch seams and sap wood will cause failure, even in coated staves. Coating not only adds to the cost of manufacture, but increases the weight materially. Machine-banded pipe, being essentially a factory product, its cost is affected greatly by freight rates, as shipment from factory to site of erection determines the economy of its use in a great many cases. In this type redwood has a distinct advantage over fir or pine, as it is unnecessary to apply an artificial coating to preserve the staves; therefore, having no coating of tar or asphalt, and well-seasoned redwood being very light, it has a very low shipping weight. The coating often serves to cover defects in material and manufacture. Fir and pine pipe should be inspected rigidly before acceptance; and redwood pipe should be left open to inspection and thus save the difference in weight and the cost of dipping. Painting the wire is useless. When the pipe is built it is impossible to be so careful in handling that none of the paint will be scraped off. The wire exposed in one spot leaves a weak place at which corrosion may start. As the wire is wound under heavy tension, there can be no protective coating on it where it touches and is embedded in the wood; for that reason only a small part of its surface is coated.

To prevent machine-banded pipe from drying out and collapsing, thorough drying of the stave material and proper winding are necessary. By the use of thoroughly dried wood, wound under heavy tension in the wire, with close spacing to draw the staves together, securely and completely, high-grade pipe is obtained. Tests in winding redwood

pipe show that a tension of 25 000 lb. per sq. in. in the wire embeds it securely in the wood and draws thoroughly dry staves together properly without crushing the fiber under the wire or along the edges of the staves. A tongue slightly longer than the groove also assists in making the pipe water-tight when exposed to the sun after the water is drawn off. Exhaustive tests at the plant of the Redwood Manufacturers Company, at Pittsburg, Cal., showed that this tension produces a pipe having greater strength, in resisting possible over-loads, than if wound under less tension, and that higher tension crushes the wood fibers. This initial tension in the wire entirely disappears after winding, and the ultimate tension is that due solely to the water pressure.

The secret of correct manufacture is the thorough seasoning of the wood. Such a pipe can be laid directly on the surface of the ground and exposed to the heat of the sun without injury. A slight tongue and groove in the sides of the staves prevents their displacement if they shrink. This should occur only to a slight extent under most severe conditions, if the staves are properly dried and a proper process of winding is used.

Kiln-dried wood should not be used for machine-banded pipe. Pitch and sap should not be allowed, nor should untreated or uncoated fir or pine be used. Redwood does not need treatment to insure a life at least as long as treated fir or pine. Fir or pine pipe should be supplied with cast-iron collars with bell hubs for caulking. On redwood pipe redwood collars may be used, and as the wood does not rely on saturation for preservation, machine-banded and continuous-stave collars may be used to advantage. Inserted joint connections may be used, but will not give as good service as collars.

In choosing a thickness of stave, it is only necessary to use that which will resist percolation successfully and can be built into a pipe. Staves must have sufficient thickness to resist the stress caused by the high tension in the wire during the process of winding. If they are of redwood, there will be no danger of decay within 15 to 25 years, which is the life of galvanized wire.

For more than 18 months the writer has investigated tests of machine-banded pipe made at the plant of the Redwood Manufacturers Company, and the results have been most startling as well as gratifying. Continued experiments on various sizes of pipes under various pressures have shown that redwood pipe made according to the specifications for

Class *A*, of the given thickness of stave, and wound with the stated sizes of wire, will be absolutely water-tight, and, if designed with a factor of safety of four against the wire breaking and a value of 800 lb. per sq. in. for bearing, it will withstand successfully a 200% over-load of the pressure for which it was designed. An 8-in. pipe, with a shell only $\frac{11}{16}$ in. thick, and wound with No. 8 wire (0.162 in. in diameter), with a spacing of $2\frac{3}{8}$ in. from center to center, to withstand a 75-ft. head, has operated successfully under a greater head than 200 ft. Further, this same pipe, wound for various pressure heads, has been connected to the boiler feed pump, in the engine-room at the factory of the Redwood Manufacturers Company, and subjected to pressures varying constantly between 15 and 75 lb., has withstood successfully, and is still operating with, over-loads as high as 80 lb., and has shown no leakage.

Wood pipe failures, when occurring in the body of the pipe, generally appear first along the longitudinal seams, which open up, allowing leakage. This is caused by the wire sinking into the wood; in other words, the bearing between the wire and the wood being destroyed, the staves move outward and the seams open. When this occurs the pipe is a failure. After such failure the staves return to their normal positions on release of the pressure, and the pipe will still operate successfully under the pressure for which it was designed.

It makes no difference whether the thickness of a stave is 1 or 2 in.; after the outer $\frac{1}{16}$ or $\frac{1}{8}$ in. has decayed, the staves move outward and the pipe fails. With thin staves, however, which can receive more perfect saturation, the maximum life is obtained.

The process of winding the wire on the pipe and drawing the wood together with the proper tension in the wire, actually determines the thickness of the staves. If the latter are too thin, they cannot be drawn into a firm seat against each other, but will buckle; the limiting thickness must be determined by experiment. During winding, a constant and uniform tension should be kept on the wire, drawing in the staves sufficiently to make all joints absolutely tight without crushing the wood. The closer the wire is spaced in this winding the better the staves are drawn together, and the tension required to do so is a minimum. A gauge, registering the actual tension in the wire, should be directly in front of the operator of the winding machine. The

tension varies, of course, with the diameter of the pipe and the spacing of the wire.

It is quite proper to give some guaranty of wood pipe design to the purchaser. When buying cast-iron or steel pipe, the head the pipe will withstand is known, but in wood pipe design and manufacture there are so many uncertainties that, without some guaranty of its strength, an engineer is at a loss to know its quality. He can check up the size of the wire and the spacing, but knows nothing as to the care in manufacture. If a pipe, guaranteed, say, for 50 or 100% over-load, can be obtained, the engineer then has a basis on which to work, and this is the ultimate method of correct manufacture to be expected. To secure a theoretical and practical basis for the design of machine-banded pipe and to determine an over-load for a guaranty, was the object of the tests just mentioned.

A radical departure from customary methods of pipe design is necessary to secure desired results. It must be remembered that, in mending a hoe handle, the farmer takes a small fine wire and wraps it as close as possible around the fractured part. He does not take a heavy wire and wind the handle with wide spacing. The correct principle of pipe winding is similar: Use small wire closely spaced, giving sufficient steel with such spacing as to insure a factor of safety of at least four against rupture under tension in the wire. A closer spacing is required on small pipe to prevent the wire from sinking into the wood because of insufficient bearing area. The use of small wire increases the bearing area between wire and wood. The reason for this can be readily understood, but the lasting quality of the small wire has to be considered.

Repeated tests and consultations with high authority on wire manufacture, and a study of the reasons, would show that the smaller wires are as well protected with galvanizing as the larger ones, and, if anything, a little better. The Western Union test, taken as a method of comparing the galvanizing on various sizes of wire, gave results which favored the smaller wires. The authorities, who are the wire manufacturers, state that the smaller wires are better galvanized because more care is taken with them in order to secure a product of the very highest grade. The reason is that, on account of the smaller wires being in greater demand, more care is taken to secure a uniform and high-grade product. In any factory output, market conditions

must be considered, and in the wire market small wire is of the best quality. Another reason for the use of small wire is the danger of destroying the galvanizing on large wire when winding pipe of small diameter. It is common to find a path of spelter directly under the pipe winding machine when winding small pipe with heavy wire, the result of the galvanizing spalling off. It is also found to spall off in splicing with the old twist, or Western Union splice. Splicing is necessary when coming to the end of the coil of wire while winding a section of pipe, and is of frequent occurrence when winding heavy-pressure pipe where close spacing is required. In making this splice the wire is twisted around its own diameter, and this injures the zinc coating.

Maximum efficiency is secured where there are no splices. Electro-welding with re-galvanizing has been tried, but the re-galvanizing is unsatisfactory, and, until better methods are obtained, splices should be eliminated. All fastenings holding wire in place on pipe should be galvanized. Pressed-steel clips will rust if not protected.

The essential feature for success in machine-banded pipe is the proper use of the proper materials. The wrong use of a good material will be as productive of failure as the use of poor material.

With high-grade lumber and a high-grade wire, with which every precaution is taken for protection against corrosion, and with scientific methods of pipe winding, thinner staves and smaller wire may be used. If reliance can be placed on the manufacturers, such methods will result in economy.

The foregoing comments and suggestions are not intended to serve as a theoretical basis for pipe manufacture, but to point out the methods in use to-day by the various manufacturers. The detrimental features are pointed out, the scientifically based principles are outlined, and the specifications in the Appendix are suggested for securing uniform practice, thus enabling engineers to know what type they will obtain when they call for bids.

The specifications should be based on the use to which the pipe will be put. No engineer would think of calling for 1:2:4 concrete for rough foundation work, where water-tightness and strength are only secondary considerations. Neither would he specify a 1:3:6 mix for concrete conduits, where both density and maximum strength are required. It is the same with wood pipe. For cheap lines of short

duration, such as construction work, when the pipe will be abandoned after a few months or a year, a low-grade uncoated fir or pine pipe may be chosen. On temporary work requiring a life of 5 or 6 years, a good grade of uncoated fir and pine pipe may well serve. For 8, 12, or 15 years' service, a good, properly painted fir pipe may be used to advantage. For permanent work, redwood should be selected, or fir or pine of high-grade staves kept saturated and well painted.

Owing to the various grades of work in which fir or pine pipes are used, alternate specifications are given, but, as redwood would only be selected for permanent work, and for some conditions in which fir or pine would fail, such as in low-pressure work, only one specification is given for this wood, and this covers the highest grade.

It is sincerely hoped that this paper may lead to a definite idea of the merits of wood pipe and the adoption of uniform specifications.

In writing specifications for wood pipe, various pipes will be designated by classes, based on the nature of the work they are to do.

Continuous-Stave Pipe.

Class A.—A pipe having a maximum life, under all conditions, and this will be 25 years when receiving no care whatsoever; a life greater than 25 years, if under continuous operation; and a probable life of 50 years, or more, if in continuous operation under at least a moderate head, if the bands are given attention and corroded ones are renewed. This includes pipe made from clear, air-dried redwood.

Class B.—This class includes coated pine or fir, in such a situation as to be open to continuous inspection, so that it may be given constant attention, comprising re-painting staves and renewing bands. This pipe will be placed under Class *A*, on theory only, as experience has not yet confirmed such an assumption.

Class C.—This class will have a maximum life of 10 years and an average life of 7 years. It will include uncoated fir, pine, or other suitable wood.

Machine-Banded Pipe.

Class A.—This class will have a life of from 15 to 25 years when receiving no attention; a longer life under ideal conditions, as when laid in soils having the least possible corrosive effect on the galvanized wire, and when operating under pressure, so as to insure complete saturation of the wood. Pipes of this class will be guaranteed to

withstand severe conditions of over-load, such as in hydro-electric work, general water-works for city supply, and high-pressure pumping lines; and will be guaranteed to withstand an over-load of 100% under test.

It will include pipes of clear, air-dried, redwood, manufactured according to the specifications in the Appendix.

Class B.—This class will have a life of at least 10 years, and a probable life not exceeding 15 years. It will include pipe made of redwood, or of coated fir or pine, etc., manufactured according to present-day standards, as indicated by the specifications covering this class.

Class C.—Pipes of this class will be used for temporary work only, and may be manufactured from redwood, fir, pine, or any other wood, with or without coating, as desired.

APPENDIX.

SPECIFICATIONS FOR CONTINUOUS-STAVE PIPE, CLASS *A*.

The staves shall be of clear, air-dried, California redwood, seasoned at least one year in the open air, and shall be free from knots (except small knots appearing on one face only), sap, dry rot, wind-shakes, pitch, pitch seams, pitch pockets, or other defects which would materially impair their strength or durability. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe; and the edges shall be beveled to true radial planes. The staves shall be milled from stock sizes of lumber, the net finished thickness of the stave, for the various diameters of pipe, shall be as given in Table 2. The ends shall be cut square and slotted to receive the metallic tongues which form the butt joints. The slots shall appear in the same position on each stave, and shall be cut to make a tight fit with the tongues in all directions. The staves shall have an average length of at least 15 ft. 6 in., and not more than 1% shall have a length of less than 9 ft. 6 in. Staves shorter than 8 ft. will not be accepted.

The metallic tongues to insert in the slots in the ends of the staves shall be made from $1\frac{1}{2}$ by $\frac{1}{8}$-in. band iron, and shall be cut $\frac{1}{16}$ in. longer than the slot in the stave, so that, after the pipe is cinched, they will penetrate the adjoining staves, thereby making a watertight joint.

The bands for pipes of large diameter shall be in two sections; those for the smaller sizes shall be in one section. The bands shall be spaced on the pipe with a factor of safety of at least four, and shall consist of round, mild-steel rods, connected with malleable-iron shoes. Either open-hearth or Bessemer steel may be used. The phosphorous content in open-hearth steel shall not exceed 0.06; in Bessemer steel it shall not exceed 0.10. The ultimate strength shall be from 55 000 to 65 000 lb. per sq. in. Steel having an ultimate strength of more than 65 000 lb. per sq. in. will not be rejected provided it shows an elongation of not less than 26% in 8 in. The yield points shall be not less than one-half the ultimate strength, and shall be determined by the drop of the beam of the testing machine. A minimum percentage in 8 in. of 1 400 000 divided by the ultimate tensile strength shall be taken as the elongation; but, the following modifications shall be made for bands less than $\frac{7}{16}$ in. and more than $\frac{3}{4}$ in. in diameter.

(*a*). For each increase of $\frac{1}{8}$ in. in diameter greater than $\frac{3}{4}$ in. a deduction of 1 shall be made from the specified percentage of elongation.

(*b*). For each decrease of $\frac{1}{16}$ in. in diameter less than $\frac{7}{16}$ in. a deduction of 1 shall be made from the specified percentage of elongation.

. The rods or bands shall be capable of bending 180° around a diameter equal to that of the specimen tested, without fracture on either side. The threads shall be cold-rolled, United States Standard; the threaded portion of the band shall have an ultimate strength equal to that required for the rods. The nut shall conform to the Colorado Fuel and Iron Company's standard* for the respective diameters, and shall be tapped so as to make a snug but easy running fit. The bands shall be provided with button heads, according to the Colorado Fuel and Iron Company's standard,* and the heads and the sections under the heads shall not fail at less than is required for the body of the rod when tested through a U-slot. One bending and two tension tests shall be made on the rods for each melt of open-hearth steel, and one bending and one tension test for each blow of Bessemer steel rolled. Two bands for each blow or melt shall be tested against the head and thread. The bands shall be subject to rejection if the actual weight of any lot varies more than 5% above or below the theoretical weight of that lot. The bands shall be free from any injurious seams, flaws, or cracks, and shall have a workmanlike finish.

The shoes shall be of the Allen type, fitting closely to the outside curvature of the pipe, and designed so that, after the bands are cinched tight, they will lie in a plane at right angles to the horizontal axis of the pipe. The shoes shall be clean castings, made from the best grade of malleable iron, free from flaws, tags, or blow-holes; and shall have a tensile strength of about 40 000 lb. per sq. in. The shoes shall be guaranteed to be stronger under test than the bands with which they are to be used.

The coating for all metal work—shoes, bands, or tongues—shall be a high-grade preservative paint, of such consistency that it will not run in hot weather or peel off in cold weather.

The coating for the bands shall be hot, and the bands shall remain in the liquid for sufficient time to insure that they will attain the same temperature as the liquid.

When a conduit is to be erected in a tropical climate, similar to that of the Hawaiian Islands, the Philippines, Central America, etc., all metal work shall be protected with red lead or galvanizing. The red lead shall be of the best quality, containing approximately 10% of litharge to insure drying without scaling. The bands, on coming from the rolls, shall be dipped in linseed oil to prevent the formation of mill scale prior to the application of the red lead. The galvanizing shall be of a standard quality, giving a full and complete coating to the metal over its entire surface.

The diameters of the rods and the maximum allowable spacing shall be as given in Table 2.

* Or other specified standard—the C. F. & I. being that generally accepted.

TABLE 2.—DETAILS OF DESIGN FOR CONTINUOUS-STAVE PIPE.

CLASSES *A*, *B*, AND *C*.

Size of pipe, in inches.	Stave Thickness, in Inches.		Number of Staves to Circle.		Stock Size of Lumber, in Inches.		Number of Feet, Board Measure, per Foot of Pipe.		Top Width of Stave, in Inches.		Size of band, in inches.	Number of pieces in band.	Spacing of Bands, in Inches.		Radius of curvature to which pipe can be built, in feet.
	Standard.	Maximum.	Standard.	Maximum.	Standard.	Maximum.	Standard.	Maximum.	Standard.	Maximum.			Maximum allowable.	For 100-ft. head.	
12	1 3/8	1 5/8	13	13	2×4	2×4	8 2/3	8 2/3	3.560	3.679	3/8	1	10	6.38	60
14	1 3/8	1 11/16	15	15	2×4	2×4	10	10	3.512	3.642	3/8	1	10	5.45	70
16	1 7/16	1 11/16	17	17	2×4	2×4	11 1/3	11 1/3	3.496	3.588	3/8	1	10	4.76	80
18	1 7/16	1 11/16	18	19	2×4	2×4	12	12 2/3	3.659	3.545	7/16	1	10	5.76	90
20	1 7/16	1 11/16	20	20	2×4	2×4	13 1/3	13 1/3	3.617	3.688	7/16	1	10	5.20	100
22	1 7/16	1 11/16	22	22	2×4	2×4	14 2/3	14 2/3	3.572	3.643	7/16	1	10	4.73	110
24	1 7/16	1 11/16	23	24	2×4	2×4	15 1/3	16	3.695	3.604	7/16	1	10	4.34	120
26	1 1/2	1 9/16	17	17	2×6	2×6	17	17	5.374	5.397	7/16	1	10	4.00	130
28	1 1/2	1 9/16	18	18	2×6	2×6	18	18	5.432	5.453	7/16	1	10	3.72	140
30	1 1/2	1 9/16	19	19	2×6	2×6	19	19	5.477	5.498	1/2	1	10	4.53	150
32	1 1/2	1 9/16	20	20	2×6	2×6	20	20	5 525	5.545	1/2	1	10	4.25	160
34	1 1/2	1 9/16	21	21	2×6	2×6	21	21	5.562	5.580	1/2	1	10	3.98	170
36	1 9/16	1 5/8	22	22	2×6	2×6	22	22	5.620	5.638	1/2	1	10	3.77	180
38	1 9/16	1 5/8	23	23	2×6	2×6	23	23	5.657	5.674	1/2	1	10	3.57	190
40	1 9/16	1 5/8	24	25	2×6	2×6	24	25	5.682	5.471	1/2	1	10	3.39	200
42	1 5/8	1 11/16	26	26	2×6	2×6	26	26	5.513	5.516	1/2	1	10	3.23	210
44	1 5/8	1 11/16	27	27	2×6	2×6	27	27	5.536	5.551	1/2	1	10	3.09	220
46	1 5/8	1 11/16	28	28	2×6	2×6	28	28	5.570	5.580	1/2	1	10	2.96	230
48	1 5/8	1 11/16	29	29	2×6	2×6	29	29	5.596	5.615	1/2, or 5/8	1	10	{2.84 4.41}	240
50	1 5/8	1 11/16	30	30	2×6	2×6	30	30	5.618	5.632	5/8	1	10	4.24	250
52	1 5/8	1 11/16	31	31	2×6	2×6	31	31	5.652	5.662	5/8	1	10	4.07	260
54	2 1/2	2 11/16	33	34	3×6	3×6	49 1/2	51	5.658	5.528	5/8	2	10	3.92	270
56	2 1/2	2 11/16	34	35	3×6	3×6	51	52 1/2	5.678	5.552	5/8	2	10	3.80	280
58	2 1/2	2 11/16	36	36	3×6	3×6	54	54	5.542	5.574	5/8	2	10	3.66	290
60	2 1/2	2 11/16	37	37	3×6	3×6	55 1/2	55 1/2	5.563	5.595	5/8	2	10	3.54	300
62	2 1/2	2 11/16	38	38	3×6	3×6	57	57	5.584	5.615	5/8	2	10	3.42	310
64	2 1/2	2 11/16	39	39	3×6	3×6	58 1/2	58 1/2	5.604	5.634	5/8	2	10	3.32	320
66	2 1/2	2 11/16	40	40	3×6	3×6	60	60	5.622	5.562	5/8	2	10	3.22	330
68	2 1/2	2 11/16	41	41	3×6	3×6	61 1/2	61 1/2	5.640	5.669	5/8	2	10	3.12	340
70	2 1/2	2 11/16	42	42	3×6	3×6	63	63	5.657	5.685	5/8	2	10	3.03	350
72	3 1/2	3 11/16	44	45	4×6	4×6	88	90	5.685	5.585	5/8, 3/4	2	8, 10	{2.95 4.24}	430
74	3 1/2	3 11/16	46	46	4×6	4×6	92	92	5.579	5.604	3/4	2	10	4.12	440
76	3 1/2	3 11/16	47	47	4×6	4×6	94	94	5.594	5.620	3/4	2	10	4.02	450
78	3 1/2	3 11/16	48	48	4×6	4×6	96	96	5.610	5.634	3/4	2	10	3.92	470
80	3 1/2	3 11/16	49	49	4×6	4×6	98	98	5.625	5.649	3/4	2	10	3.81	480
82	3 1/2	3 11/16	50	50	4×6	4×6	100	100	5.640	5.663	3/4	2	10	3.72	490
84	3 1/2	3 11/16	51	51	4×6	4×6	102	102	5 654	5.677	3/4	2	10	3.63	500
86	3 1/2	3 11/16	52	53	4×6	4×6	104	106	5.667	5.583	3/4	2	10	3.55	516
88	3 1/2	3 11/16	53	54	4×6	4×6	106	108	5.681	5.596	3/4	2	10	3.48	528
90	3 1/2	3 11/16	55	55	4×6	4×6	110	110	5.588	5.609	3/4	2	10	3.39	540
92	3 1/2	3 11/16	56	56	4×6	4×6	112	112	5.602	5.623	3/4	2	10	3.32	552
94	3 1/2	3 11/16	57	57	4×6	4×6	114	114	5.616	5.637	3/4	2	10	3.25	564
96	3 5/8	3 11/16	58	58	4×6	4×6	116	116	5.642	5.649	3/4	2	8	3.18	575
108	3 5/8	3 11/16	65	65	4×6	4×6	130	130	5.620	5.626	3/4	2	8	2.83	750
120	3 5/8	3 11/16	71	72	4×6	4×6	142	144	5.681	5.614	3/4	2	8	2.54	1 080
132	3 5/8	3 11/16	78	78	4×6	4×6	156	156	5.659	5.669	3/4	2	8	2.32	1 190
144	3 5/8	3 3/4	85	85	4×6	4×6	170	170	5.642	5.651	3/4, 7/8	2	8, 6	{2.12 2.89}	1 300

SPECIFICATIONS FOR CONTINUOUS-STAVE PIPE, CLASS *B*.

The staves for uncoated redwood pipe shall be the same as those specified for Class *A*, continuous-stave pipe.

The staves for coated pipe shall be of yellow fir (Douglas fir), redwood, or such other wood, acceptable to the engineer, as may be specified by the bidder at the time of submitting his proposal. The wood shall be sound, straight-grained, and free from dry rot, pitch seams, pitch pockets, checks, wind-shakes, bruised ends, sap wood, and other imperfections which would impair its strength or durability. Through knots or knots at the ends or edges of staves will not be allowed. Sound knots and knots not exceeding $\frac{1}{2}$ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned by air-drying for not less than 60 days, in open piles, or by thorough kiln-drying. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe; and the edges shall be beveled to true radial planes. The ends shall be cut square and slotted to receive the metallic tongues which form the butt joints. The slots shall appear in the same position on each stave, and shall be cut to make a tight fit with the tongues in all directions. The staves shall have an average length of at least 16 ft., and not more than 1% shall have a length of less than 9 ft. 6 in. Staves shorter than 8 ft. will not be accepted. The specifications for the tongues, rods, and shoes, and for the coating of the metal work shall be the same as for Class *A* pipe.

Redwood pipe need not be protected with any artificial coating. Pipe made of fir or other wood shall be coated. This coating shall be continuous and heavy; it shall be not less than $\frac{1}{16}$ in. thick, and shall consist of more than one individual coat of a mixture of asphaltum and tar. The first coating shall be allowed to dry thoroughly before the application of the second. The coating shall be hard, tough, durable, perfectly water-proof, and strongly adhesive to the metal and the staves. It shall show no tendency to flow under a summer temperature, and shall not become brittle, so as to crack or scale, under a freezing temperature. The coating shall be well spread and rubbed in with brushes, or shall be applied as a spray under pressure; but, in either case, all cracks, checks, or other surface irregularities shall be thoroughly covered and filled.

SPECIFICATIONS FOR CONTINUOUS-STAVE PIPE, CLASS *C*.

The staves shall be of Douglas fir, redwood, or other wood acceptable to the engineer. The wood shall be sound, straight-grained, free from dry rot, checks, wind-shakes, and other imperfections which would impair its strength or adaptability for pipe construction. Sap will

not be allowed on more than 10% of the inside face of any stave, and in not more than 10% of the total number of pieces. The sap shall be bright, and shall not occur within 4 in. of the end of any piece. Pitch seams will be permitted in not more than 10% of the total number of pieces, if showing on the edge only and if not longer than 4 in. or wider than $\frac{1}{16}$ in. Through knots or knots at the edge or within 6 in. of the ends of the staves will not be allowed. Sound knots not exceeding $\frac{1}{2}$ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned by air-drying for not less than 60 days, in open piles, or by thorough kiln-drying. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe; and the edges shall be beveled to true radial planes.

The remainder of the specifications are as outlined for Class *B* pipe, except that the coating of the pipe may be omitted.

Specifications for Machine-Banded Pipe, Class *A*.

The staves shall be of clear, air-dried, California redwood, seasoned at least one year in the open air, and shall be free from knots (except small knots appearing on one face only), sap, dry rot, windshakes, pitch, pitch seams, pitch pockets, or other defects which would materially impair their strength or durability. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe; the edges shall be beveled to true radial planes, and shall also have a small tongue and groove. After the staves are built up in sections, the ends shall be cut square, and a smooth tenon shall be turned on the end of each section, to make a tight fit with the collars or couplings. The sections shall be in random lengths of from 8 to 20 ft. The staves shall be milled from stock sizes of lumber. The net finished thickness of the stave, for the various diameters of pipe, shall be as given in Table 3.

Both pipe and wire-wound collars, when such are used, shall be wound spirally with a heavily galvanized steel pipe-winding wire. This wire shall be of such a size, and spaced at such a distance (according to the head under which the pipe will operate), as to give a factor of safety of at least four against breaking. This wire shall also be of such a size, and spaced at such a distance, as to give a bearing surface which will make the pipe safe against failure by the wire sinking into the wood under pressure, which might cause the pipe to leak along the longitudinal joints. The sizes and spacing of the wire for the various sizes of pipe operating under different pressure heads shall be as given in Table 3.

The ends of the wire shall be fastened securely to the pipe with pressed-steel or malleable-iron clips, which shall be protected against

TABLE 3.—MACHINE-BANDED PIPE.

Diameter of pipe, in inches.	Thickness of staves, in inches.	No. of wire gauge.	SPACING OF WIRE, IN INCHES, FOR VARIOUS HEADS, IN FEET.															
			25	50	75	100	125	150	175	200	225	250	275	300	325	350	375	400
CLASS *A*.																		
2	7/8	12	3½	3½	2 7/16	1 13/16	1 7/16	1 3/16	1 1/16	15/16	13/16	¾	11/16	⅝	9/16	½	½	7/16
3	7/8	12	3½	3 1/16	2 1/16	1⅜	1¼	1	⅞	¾	11/16	⅝	9/16	½	½	7/16	7/16	⅜
4	7/8	12	3½	2¾	1⅞	1⅝	1⅛	15/16	13/16	11/16	⅝	9/16	½	7/16	7/16	⅜	⅜	5/16
5	7/8	12	3½	2⅜	1⅝	1 3/16	15/16	13/16	11/16	⅝	½	½	7/16	⅜	⅜	5/16	5/16	5/16
6	7/8	10	3½	3¼	2⅛	1⅝	1¼	1 1/16	15/16	13/16	11/16	⅝	9/16	½	½	7/16	7/16	⅜
8	15/16	8	3½	3½	2 7/16	1 13/16	1 7/16	1¼	1 1/16	⅞	13/16	¾	11/16	⅝	9/16	½	½	7/16
10	15/16	8	3½	2 13/16	1⅞	1 7/16	1⅛	15/16	13/16	11/16	⅝	9/16	½	½	7/16	⅜	⅜	5/16
12	15/16	8	3½	2⅜	1½	1 3/16	15/16	¾	11/16	9/16	½	7/16	7/16	⅜	⅜	5/16	5/16	5/16
14	1⅛	8	3½	2	1 5/16	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
		6	.....	.....	.....	.....	1⅛	15/16	13/16	11/16	⅝	9/16	½	7/16	7/16	⅜	⅜	5/16
16	1⅛	6	3½	2⅛	1½	1¼	1	13/16	11/16	⅝	.....	.....	.....	.....	.....	.....	.....	.....
		4	.....	.....	.....	.....	.....	.....	.....	.....	¾	11/16	⅝	9/16	½	7/16	7/16	7/16
18	1⅛	6	3½	2⅛	1⅜	1 1/16	⅞	11/16	⅝	½	.....	.....	.....	.....	.....	.....	.....	.....
		4	.....	.....	.....	.....	.....	.....	.....	.....	⅝	9/16	½	½	7/16	⅜	⅜	⅜
20	1⅛	6	3½	2	1⅛	1	¾	⅝	9/16	½	.....	.....	.....	.....	.....	.....	.....	.....
		4	.....	.....	.....	.....	.....	.....	.....	.....	9/16	½	½	7/16	⅜	⅜	⅜	5/16
22	1⅛	6	3½	1¾	1 3/16	⅞	11/16	9/16	½	7/16	.....	.....	.....	.....	.....	.....	.....	.....
		4	.....	.....	.....	.....	.....	.....	.....	.....	½	½	7/16	⅜	⅜	5/16	5/16	5/16
24	1⅛	6	3	1⅝	1	13/16	⅝	½	7/16	⅜	.....	.....	.....	.....	.....	.....	.....	.....
		4	.....	.....	.....	.....	.....	.....	.....	.....	½	7/16	⅜	⅜	⅜	5/16	5/16	¼
CLASSES *B* AND *C*.																		
2	1	8	4	4	4	3	2⅜	2	1 11/16	1½	1 5/16	1 3/16	1 1/16	15/16	⅞	13/16	¾	11/16
3	1	8	4	4	3 5/16	2½	2	1⅝	1⅜	1¼	1 1/16	1	⅞	13/16	¾	11/16	⅝	⅝
4	1⅛	6	4	4	3⅝	2¾	2¼	1 13/16	1 9/16	1⅜	1 3/16	1 1/16	1	⅞	13/16	¾	11/16	11/16
5	1⅛	6	4	4	3 7/16	2 9/16	2 1/16	1 11/16	1 7/16	1¼	1⅛	1	15/16	13/16	¾	11/16	11/16	⅝
6	1⅛	4	4	4	3¾	2 13/16	2¼	1⅞	1⅝	1⅜	1¼	1⅛	1	15/16	⅞	13/16	¾	¾
8	1¼	4	4	4	3 9/16	2 11/16	2 3/16	1¾	1½	1 5/16	1 3/16	1 1/16	15/16	⅞	13/16	¾	11/16	⅝
10	1¼	4	4	4	3 7/16	2 9/16	2 1/16	1 11/16	1 7/16	1¼	1⅛	1	15/16	13/16	¾	11/16	11/16	⅝
12	1¼	2	4	4	3⅞	2 15/16	2 5/16	1 15/16	1⅝	1 7/16	1¼	1⅛	1 1/16	15/16	⅞	13/16	¾	11/16
14	1⅜	2	4	4	3 9/16	2 11/16	2⅛	1¾	1½	1 5/16	1 3/16	1 1/16	1	⅞	13/16	¾	11/16	11/16
16	1⅜	1	4	4	3⅝	2¾	2 3/16	1 13/16	1 9/16	1⅜	1 3/16	1 1/16	1	15/16	13/16	¾	¾	11/16
18	1⅜	1	4	4	3¼	2 7/16	1 15/16	1⅝	1⅜	1 3/16	1 1/16	15/16	⅞	13/16	¾	11/16	⅝	⅝
20	1½	1	4	4	2⅞	2 3/16	1¾	1½	1¼	1 1/16	15/16	⅞	13/16	¾	11/16	⅝	9/16	9/16
22	1½	1	4	4	2⅝	2	1 9/16	1 5/16	1⅛	1	⅞	13/16	¾	11/16	⅝	9/16	½	½
24	1½	1	4	3⅝	2 7/16	1 13/16	1½	1 3/16	1	⅞	13/16	¾	11/16	⅝	9/16	½	½	7/16

corrosion by galvanizing or Sherardizing, and there shall be no splices in the wire on any section of pipe or collar. The wire shall have a tensile strength of from 60 000 to 65 000 lb. per sq. in., and an elastic limit of not less than 50% of the ultimate strength, and shall be wound under sufficient tension to be firmly seated in the wood without crushing the fibers. The tension under which the wire is wound on the pipe shall be between 25 000 and 30 000 lb. per sq. in. All wire used in the manufacture of machine-banded pipe shall withstand a test for condition of galvanizing; this test shall consist of three of the four immersions in the testing solution, specified as the Western Union test, which reads as follows:

"Method of Testing.—Samples of wire, previously cleaned with gasoline or benzine, shall be immersed to a distance of at least 4 in.

in a glass vessel containing not less than one pint of the standard solution, and allowed to remain for one minute. They shall then be removed, washed in clear water, and wiped dry with soft cotton cloth or waste. This process shall be repeated three times, making four immersions in all."

A saturated solution of sulphate of copper, having a specific gravity of 1.186 and a temperature of 65° Fahr., shall be taken as the standard solution. The temperature of the solution during the test shall not be above 68° nor below 62° Fahr. If a bright copper deposit appears on the steel after the fourth immersion, thus indicating that the wire is exposed, the galvanizing represented by the samples shall be considered faulty. Three of these immersions without showing signs of copper, shall be considered as the test for the pipe winding wire.

Wooden collars or other couplings shall be furnished under the following specifications:

Continuous-stave and machine-banded collars shall be made in the same manner as the pipe, the staves being 6 or 8 in. long, depending on the diameter of the pipe. Pipe from 2 to 6 in. in diameter, inclusive, shall have collars 6 in. long; pipes of larger diameters shall have collars 8 in. long. The continuous-stave collar shall be banded with ⅜-in. round, mild-steel rods, held together by straight-pull, malleable pipe shoes, and a nut. The machine-banded collar shall be wound with the same wire as the pipe, and in the same manner, but the wire shall be spaced closer in order to make it stronger than the pipe.

For the inserted joint connection, each section of pipe shall be mortised and tenoned with a male and female joint.

Inserted joint connections shall be used on pipe operating under pressures not exceeding a static head of 25 ft. and in sizes up to and including pipe of 12 in. inside diameter. All other sizes, and the aforementioned sizes under higher heads, shall have machine-banded collars when operating under static water pressure, or under a pumping pressure in which there will be no excessive pulsations. In all cases, machine-banded collars shall be preferred to the inserted or slip joint. On pipes having an inside diameter of 12 in. or greater, in pumping lines for hydro-electric work, where over-load, pulsation, or hammer are very likely to occur, continuous-stave collars shall be used.

All pipe shall be guaranteed to withstand an over-load of at least 50% when operating in flow lines not subject to over-load strains of any kind, and shall be guaranteed to withstand a 100% over-load when operating under all other conditions.

Specifications for Machine-Banded Pipe, Class *B*.

The staves shall be of clear, air-dried redwood, uncoated, or of fir protected by a coating having an asphaltic base, and rolled in saw-dust.

The staves for uncoated redwood pipe shall be the same as specified for Class *A* machine-banded pipe.

The staves for coated pipe shall be of yellow fir (Douglas fir), redwood, or such other wood, acceptable to the engineer, as may be specified by the bidder at the time of submitting his proposal. The wood shall be sound, straight-grained, and free from dry rot, pitch seams, pitch pockets, checks, wind-shakes, bruised ends, sap wood, and other imperfections which would impair its strength or durability. Through knots or knots at the ends or edges of staves will not be allowed. Sound knots and knots not exceeding ½ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned by air-drying for not less than 60 days, in open piles, or by thorough kiln-drying. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe; the edges shall be beveled to true radial planes, and shall be provided with a small tongue and groove. After the staves are built up in sections, the ends shall be cut square, and a smooth tenon shall be turned on the end of each section, to make a tight fit with the collars or couplings. The sections shall be in random lengths of from 8 to 24 ft., the limits for redwood pipe being from 8 to 20 ft. and for fir pipe from 8 to 24 ft.

The thickness of the staves of redwood pipe shall be the same as specified for Class *A* pipe.

For pipe of fir and other woods the thickness of the staves shall be as given in Table 3.

The pipe shall be wound spirally with a special, heavily galvanized, steel pipe-winding wire, and spaced with a factor of safety of four. The size of the wire shall depend on the diameter of the pipe and the pressure. Further, the wire shall be of such a size, and spaced at such a distance, as to insure the pipe against failure by the wire sinking into the wood and allowing the longitudinal seams to open.

The ends of the wire shall be fastened securely with pressed-steel or malleable-iron clips, which shall be protected against corrosion by galvanizing or Sherardizing. The wire shall have a tensile strength of from 50 000 to 65 000 lb. per sq. in., and an elastic limit of not less than 50% of the ultimate strength, and shall be wound under sufficient tension to be firmly seated in the wood without crushing the fibers.

Redwood pipe may be furnished with connections of the same type as specified for Class *A* pipe. If fir or other woods are used, cast-iron collars shall be provided for connecting sections of pipe. These cast-iron collars shall be of pure, gray iron, of the highest grade, free from tags, blow-holes, or other imperfections which would impair their strength. The inserted joint connections may be used under the same conditions as specified for Class *A* pipe, but shall be heavily

coated with protective compound after erection. If wire-wound collars or continuous-stave collars are used, they shall be made of redwood staves.

Redwood pipe need not be protected with any artificial coating. Pipe made of fir or other wood shall be coated. This coating shall be continuous and heavy; it shall be not less than $\frac{1}{16}$ in. thick, and shall consist of more than one individual coat of a mixture of asphaltum and tar. The coating shall be hard, tough, durable, perfectly water-proof, and strongly adhesive to the metal and the staves. It shall show no tendency to flow under a summer temperature, and shall not become brittle, so as to crack or scale, under a freezing temperature. The pipe shall be hot-dipped, and its tenoned ends shall be protected during the dipping so as to prevent the mixture from getting on the inside of the pipe. After the pipe has been dipped it shall be rolled down an incline covered with fine saw-dust in order to cover it and enable it to be handled without the coating sticking to surfaces with which it comes in contact. The guaranties shall be the same as those applying to Class *A* pipe.

Specifications for Machine-Banded Pipe, Class *C*.

The staves shall be of Douglas fir, redwood, or other wood acceptable to the engineer. The wood shall be sound, straight-grained, and free from dry rot, checks, wind-shakes, wane, and other imperfections which would impair its strength or adaptability for pipe construction. Sap will not be allowed on more than 10% of the inside face of any stave, and in not more than 10% of the total number of pieces. The sap shall be bright, and shall not occur within 4 in. of the end of any piece. Pitch seams will be permitted in not more than 10% of the total number of pieces, if showing on the edge only and if not longer than 4 in. or wider than $\frac{1}{16}$ in. Through knots or knots at the edge or within 6 in. of the ends of the staves will not be allowed. Sound knots not exceeding $\frac{1}{2}$ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned by air-drying for not less than 60 days, in open piles, or by thorough kiln-drying. The sides of staves shall be milled to conform to the inside and outside radii of the pipe; the edges shall be beveled to true radial planes, and shall be provided with a small tongue and groove. After the staves are built up in sections, the ends shall be cut square, and a smooth tenon shall be turned on the end of each section, to make a tight fit with the collars or couplings. The sections shall be in random lengths of from 8 to 24 ft.

Both pipe and wire-wound collars, when such are used, shall be wound spirally with a heavily galvanized steel pipe-winding wire, and spaced with a factor of safety of not less than four. The wire shall

also be spaced for each diameter so as to insure the pipe against failure by the wire sinking into the wood and allowing the longitudinal seams to open.

The ends of the wire shall be fastened securely with pressed-steel clips. The wire shall have a tensile strength of from 60 000 to 65 000 lb. per sq. in., and an elastic limit of not less than 50% of the ultimate strength, and shall be wound under sufficient tension to be firmly seated in the wood without crushing the fibers.

Inserted joint connections may be used for a pressure of 100 ft. or less; on higher pressures wire-bound collars shall be used. All pipe shall be guaranteed to withstand a 50% over-load of the pressure for which it is designed.

DISCUSSION

HERMANN VON SCHRENK,* ASSOC. AM. SOC. C. E. (by letter).—This paper has been read with much interest by the writer, but he finds that it contains certain rather startling errors relating to cypress, its method of growth, and its grade. Mr. von Schrenk.

On page 440 the author states: "A cypress tree is the product of four or five small trees growing together." Furthermore, he explains that, as a result of such growth, the sapwood does not occur around the circumference of the tree, but in streaks throughout the trunk. This is so startling an error that it should not go unchallenged. The cypress does not grow as stated by the author, but like all other trees. It starts from a seed, and develops a trunk as it grows older; and the sap ring, under all circumstances and conditions, is on the outside, immediately under the bark, just as in practically all other trees. Having studied the growth of these trees for about 25 years, under every conceivable condition, the writer has yet to find a single instance of any such occurrence as is mentioned in the paper. Individual trees formed by the union of several sprouts are possible, of course, but these would be more likely to occur in redwood than in cypress, because, unlike the redwood, the cypress does not sprout from the stump. The possible reason for Mr. Partridge's mistake is indicated in his discussion as to sap streaks in the wood. He states: "It is common to see clear cypress with yellow sap streaks running through the center at intervals of about 4 in." From this sentence, it would appear that the author interprets the lighter streaks of color in the heartwood as sap streaks. The color of the cypress heartwood varies from very pale yellow to almost black, and this may occur in one tree or in different trees growing in the same region. This variation in color has given rise to all sorts of common names, such as white cypress, yellow cypress, black cypress, etc., and it is very common to find these curious color differences or markings in the same trunk. These color variations, however, have nothing to do with the sapwood. There is never any question about identifying the sap ring, which in cypress is usually very narrow, particularly in the older trees. The curious color variations or streaks in the cypress have been ascribed to the variation in absorption of material from the soil. The writer has frequently noted that in certain regions one type of color variation will appear and in other regions another type. The writer would be much interested to have a case pointed out where a trunk actually is composed of more than one original seedling, because this certainly would be a scientific curiosity. If Mr. Partridge's statements were true, what would become of the bark on the inside, if several trees fused?

* St. Louis, Mo.

Mr. von Schrenk.

Attention is also called to a slight error in Mr. Partridge's description of grades. Although it is true that, in the listing of the grades by the Southern Cypress Manufacturers' Association, the grade "Tank" appears first in the book of specifications, as a matter of fact Grade "A" is really higher than "Tank." In cypress there is no such grade known as "Clear." The manufacturers of cypress regard the Grade "A", in the heavier thicknesses, as being better than the "Tank" grade in the same thickness, for the reason that knots are eliminated on the face side of "A", whereas any number of water-tight knots are admitted in the grade of "Tank."

Mr. Partridge is probably correct in his surmise that 4-in. "Tank" stock is better material than 4-in. Grade "A" for pipe staves, for two reasons: First, an occasional piece of Grade "A" would have a very slight quantity of sap on the reverse side, whereas "Tank" would be strictly free from sap, although it would contain knots. Second, the "Tank" stock, in the heavier thicknesses, can be purchased at a lower price than the Grade "A" of the same thicknesses. As indicated by Mr. Partridge, the mere presence of knots does not influence in any way the value of "Tank" stock for stave purposes.

Attention is also called to a slight variation in the author's nomenclature from that generally accepted as standard by such organizations as the American Society for Testing Materials, the American Railway Engineering Association, and the Forest Service and other Government bureaus. The terms usually applied as standard for the two grades of wood formed by a tree during the year are "springwood", for the soft wood formed early in the year, and "summerwood", for the hard wood formed later in the year. Many readers will doubtless confuse Mr. Partridge's names of "summerwood" and "winterwood" for the more usual terms of "springwood" and "summerwood."

The writer would like to ask the author for a further explanation as to why penetration is insisted on for a stave. The writer may not understand the meaning of the word "penetration", but assumes that it means the thorough absorption by the stave of water or whatever liquid is passed through the pipe. If any figures are available, as the result of tests, it would be of interest to know on what basis the statement is made that a dense piece of pine, for instance, will not make as good a pipe as a piece of redwood, or other material, of similar density.

Mr. Bell.

FRANK F. BELL,* ESQ. (by letter).—The following general ideas along the lines of this paper are based on the writer's experience in wood stave pipe design and construction.

The writer finds it to be generally true, as Mr. Partridge has outlined, that there is little authentic knowledge among engineers relative to either continuous-stave or machine-banded pipe, as it is considered

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somewhat of a specialty, although coming more and more into use in modern engineering practice. Mr. Bell.

Ideas regarding such pipe are at variance in different parts of the country. In general, the specifications and design are under the influence of the locality in which the engineer intends to build. In the Northwest, we find strong adherents to pipe made from Oregon pine and Douglas fir; in the Southwest, redwood; and in the East, Canadian pine and yellow pine. This is due mostly to the sales talk of the representatives of the various manufacturers of pipe, and the engineer has to be guided more or less by what he hears rather than by actual experience or standards. Wood stave pipe has passed the stage where it can be considered as a specialty; and it is certainly time that general practice and standard specifications should be worked out.

The author's presentation of the subject is complete, but, of course, with reference to the specifications, there may be more or less disagreement. As a start, however, the writer believes the paper will furnish a reasonable basis for the best practice.

It is generally admitted that, for purposes of durability, redwood is logically the best. The writer does not believe there will be any discussion on this point. He has heard engineers criticize this lumber on account of its softness, but, for wood pipe, such criticism is hardly worth considering, as the strength depends on the band spacing, and when the pipe is made up, there seems to be no material difference in rigidity when redwood and harder woods are compared.

The author does not mention yellow pine as a good lumber for wood stave pipe, but this is hardly worthy of important notice, as such wood is too scarce to be a logical material, and would be put in a class with cypress.

In the eastern part of the United States, the general practice is to use thick staves and flat bands for machine-made pipe, but in the West round steel wire is used. A selection between these two styles should be based on local requirements. For continuous-stave pipes, the eastern style is not in keen competition. It is mostly in specifications for pipe of large diameter that standard practice is needed.

The writer agrees with Mr. Partridge that redwood need never be given a coat of preservative, provided clear lumber is used, according to his specifications. On the other hand, in pipe constructed of other kinds of lumber, a preservative coating is necessary, if longevity is wanted. Redwood, under all general conditions, will easily outlast bands and wire winding, without any preservative application.

The writer is familiar with the North Yakima development, mentioned in the paper, as well as the sewer at Palo Alto, Cal. The North Yakima pipe underwent about as rigid a test for durability as would be found in any practice, and, from studying this particular case, it appears that the drying of the lumber is one of the most important

Mr. Bell. factors. The process of kiln-drying *versus* air-drying is, of course, familiar enough to all engineers; but the writer again agrees with Mr. Partridge that, if possible, air-dried lumber should be used, although very good results are obtained with kiln-dried fir.

In the matter of machine-banded pipe, the drying is even a more important factor, because there is no chance to "cinch up" in case of shrinkage, and the strength and durability depend on the bearing surface and winding tension of the wire.

The author's remarks on the importance of butt joints should also be emphasized, as in coutinuous-stave pipe this is generally where trouble starts first, and it is also one of the drawbacks in having wood pipe built by men who do not understand the finer details of construction.

The writer knows of a case (which happened within the last few months) of a contractor who had his own staves milled by the ordinary process, bought his own bands, and attempted to put in his own pipe according to his idea as to specifications and methods. This pipe, no doubt, will be unserviceable in a very short time, and, consequently, engineers who come in touch with this particular case will be influenced against wood pipe in general. This is why the writer believes that the author's remarks should be given favorable consideration. If standard specifications and practice are put in the hands of engineers, there will be a much more favorable attitude in general toward wood pipe, and it will be given the place it deserves as an engineering utility.

Mr. Partridge's classifications are very fair for all kinds of pipe. Naturally, the redwood man claims that his pipe is the best, the fir man his, the pine man his. The only way for the engineer to decide is to make a comparison of numerous structures. On the basis of data gathered from a number of wood stave pipes, it is thought that Mr. Partridge has rounded out the matter very well. The specification tables in the paper would be what the writer would use if he were designing a pipe line.

The writer has found by experience in many cases that a wood stave pipe has cost the designing engineer much more than it would if he had used the most economical specifications; and, in a majority of cases, the engineer has lost, in the long run, by sacrificing price to permanency, by failing to adopt the specifications and the style of pipe needed for the conditions under which he is designing, having been talked into "something just as good."

If careful unprejudiced consideration is given to this paper, engineers will feel that standards should be generally recognized, and that the proper styles, classifications, and lumbers should be given level-headed consideration. As it is, all are depending too much on the influence of representatives who are talking up the various kinds of pipe, and, naturally, where one lumber predominates, the designing engineer will be influenced in his specifications.

D. C. HENNY,* M. AM. SOC. C. E. (by letter).—From a report by the writer to the U. S. Reclamation Service in July, 1915, on the life of wood pipe, the author has quoted a tabulation in which an attempt was made to condense the information collected from past experience and special investigation. It is well to quote also from the two paragraphs in the report immediately preceding this tabulation, which read as follows: Mr. Henny.

"The investigation has had in view especially the life of pipe as affected by the durability of the wood. * * * The information collected is based mostly on reports received from managers or owners, and in small part on personal observation. It is not as complete as is desirable, which, however, is not due to failure to elicit further information.

"Reviewing the information as grouped under its headings, it may be estimated that under conditions of continuous water pressure the life of various kinds of pipe may be as follows:"

The last item of the tabulation refers to the estimated life of wood in fir pipe, well-coated, buried in loose soil, and maintained under continuous water pressure. Some additional information has come to hand since the date of the report, which, although not directly applicable, may be considered of some importance.

During October, 1916, the writer made an examination of wood pipe lines—part of an irrigation system in the vicinity of Pasco, Wash. —constructed in the spring and early summer of 1910. The pipe portion of the system consists of 4½ miles of continuous-stave, uncoated, fir pipe, 30 and 36 in. in diameter, and 12½ miles of machine-banded, coated, fir pipe, from 10 to 24 in. in diameter.

Decay in the uncoated pipe had made it necessary to rebuild 8% of the line during 1915 and 23% during 1916. Examination in October of that year showed that serious decay had then affected the remaining 69% of the pipe. It had attacked principally the bastard grain wood and wood with a coarse vertical grain, such as is derived from the top of the tree. No serious decay was found in the close-grained wood with vertical grain, and staves of this kind had been used in the reconstruction.

This is stated merely as incidental to experience with machine-banded pipe, repairs on which, up to the time of examination, had been confined to the replacement of wood sleeves. Only a small percentage of this pipe was open to inspection. This, however, showed that some of the wood in the pipe itself had been attacked, the decay being confined largely to individual staves in the upper half of the pipe. To what extent the condition of the small portion of pipe examined may be indicative of that of the remainder is problematical.

* Portland, Ore.

Mr. Henny. It was noted that, in several places where decay had occurred, there were in evidence hundreds of insects resembling ants. Whether these were cause or effect could not be determined.

All the pipe is buried in loose, sandy soil. The winter climate is mild, and the summer is hot, the temperature frequently rising above 100 degrees. The coating appeared to be the manufacturers' standard, consisting of asphalt and tar.

This pipe is not included under the conditions predicated in the tabulation quoted, as during 5 months of the non-irrigating season it is not maintained full. It is probable, however, that this fact is only in small part responsible for the decay observed. It will be necessary to examine a larger portion of this pipe, or await the results of experience with it, before dependable conclusions can be drawn. Nevertheless, its early decay in spots affords ground for reducing the life estimate, of well-coated fir pipe under very unfavorable conditions, from the figures given in the tabulation to 10 years or even less.

The author has presented his case with clearness and ability, and the virtues of redwood in pipe construction are well brought out.

Although the paper mentions the variability of wood of the same species, it does not specifically deal with the variations to which redwood itself is subject. The wood in the lower 30 ft. of a large redwood tree is exceedingly close-grained. It sinks in water, and is remarkable for its long life. Fence posts made of it have lasted more than 50 years.

Higher in the tree, the grain gradually becomes coarser, and near the top it shows fewer than ten rings to the inch. It also becomes lighter, and, although no specific tests are known to the writer, it is probable that the life of redwood as well as fir becomes shorter as the grain becomes coarser. In the past, specifications have sometimes called for sinker redwood, but, from a practical mill standpoint, this is almost impossible to supply in large quantities. However, in scientifically drawn specifications, it may be in point to limit the coarseness of the grain, the more so as the superior lightness of the coarse-grained wood offers a temptation for its selection (where freight charges become an important item), to a contractor not concerned about the ultimate life of the pipe.

In the Northwest, some makers of wood pipe are now equipped to furnish creosoted or otherwise treated fir staves. The writer sees no reason for not advancing pipe built of fir properly treated from Class *B* to Class *A* of the author's specifications, although sufficient time has not yet elapsed to demonstrate fully either the complete correctness of this or the most economical treatment available for producing the desired result.

The proper creosoting of pipe staves is not only expensive in itself, but also adds heavily to the freight charges and, to some extent, to the

Mr. Henny.

cost of pipe erection. In some sections of the country such pipe appears to have competed successfully with untreated redwood.

It is also as yet an open question as to what is the life of exposed fir pipe, and to what extent its life is extended by surface coating. The only case of decay of such pipe, which has come to the writer's attention is that of a pipe built on saddles in Utah (listed as No. 16 in the writer's report), and this was after 12 years of service. This pipe was not coated, and other uncoated pipes of this kind showed no decay after 14 years; and the writer has found, in the New England States, sound stave pipe, built probably of native pine, and resting on piers, in regard to which the time of construction reached back beyond the memory of any one who could furnish information.

On the other hand, the author has noted decay in exposed redwood pipe in Southern California, maintained full only during the irrigation season, and less than 15 years old.

The author's statement regarding the superiority of air-drying over kiln-drying is undoubtedly true, so far as past commercial methods are concerned. It is understood, however, that the subject of kiln-drying has been under scientific investigation for several years, and it is not improbable that commercially feasible methods will be perfected which will be free from the objections of wood injury to which reference was made.

So far as continuous-stave pipe is concerned, there is no serious disadvantage in the use of lumber not fully dry, and a rigid adherence to the requirement of one year's air seasoning may tend to place a serious limit on reasonable competition, or even make it impossible to obtain the material within the time required.

The author believes that the use of inserted or slip-joint pipe is not to be recommended. The only pipe of this kind examined by the writer was made of fir and, so far as the joints are concerned, was entirely successful. Pipe of this kind on the Sunnyside Project of the Reclamation Service successfully withstands a pressure of 150 ft. No injury was noted in handling or laying, contrary to the author's fears, which, however, may apply more specially to the weaker and softer redwood.

It is noted that, in the classification suggested by the author, Class *A* pipe is to have a life of 25 years or more under all conditions. On further thought he will undoubtedly desire to place some limitation on these conditions, as he must be aware of several instances of buried and exposed redwood pipe which, owing to the fact that it is not completely water-filled at all times, in combination possibly with the use of coarse-grained wood, has shown a comparatively short life.

Some 30 years ago, when continuous-stave pipe was first introduced into the Western States by the late Charles P. Allen, of Denver, Colo., it was looked on by most engineers as of doubtful utility. It has gradually established itself as a legitimate type which can be adapted to

Mr. Henny. relatively permanent or temporary uses, as requirements or financial advantage may dictate.

Experience as to its life and usefulness is fast accumulating, and renders it possible to proceed with a steadily growing sense of security as to results, to which the able exposition of fundamental principles by the author is a valuable addition.

Mr. Rust. HENRY P. RUST,* M. AM. SOC. C. E.—This is a most opportune time to advocate the use of wood in place of steel for all construction purposes, wherever possible, with due consideration for safety; and, from the title of this paper, the speaker was most hopeful that it would fill a much needed want. However, on closer examination, the paper is most disappointing, as it is evidently an attempt to advocate the use of redwood in place of Douglas fir for wood pipes, and it does not offer proper proofs for such statements as "Douglas fir and pine are fundamentally inferior to redwood."

Although, in the early days of the use of such pipe in the West, redwood was practically the only material considered, recently, Douglas fir has been more generally used. On this account the highly desirable object of such a paper will be defeated, and engineers having independent experience and realizing the circumstances will hardly treat it seriously. It will be unfortunate if the discussion turns merely into a contention between the makers of redwood and Douglas fir pipe, as is likely, because the choice of material is not the most important feature, and, under proper conditions for any wood pipe, either of these woods should be satisfactory.

There is no doubt that redwood will resist decay much better than Douglas fir or long-leaf pine, when used in any location where it will not be saturated with water; but it has always been considered a condition necessary to the successful use of wood pipe that the wood be kept saturated and the pipe full of water practically all the time. This is really not a difficult condition to meet, and will not cause the rejection of wood pipe in many cases where it might otherwise be used. If the pressure is too low or the pipe cannot be kept filled, a different construction can probably be used, which will give better satisfaction, with a comparatively lower first cost. The contention, however, that even redwood pipe should be used where the staves are not to be kept saturated practically all the time can hardly be sustained, and although it may have proved satisfactory in some such cases as the author notes, there are many other cases where it has decayed from this cause, although they have not been mentioned in the paper.

There are other causes of failure besides decay, and there are objections to the use of redwood. The wood is softer and much more

* New York City.

brittle than pine or fir. It cannot stand as great compression, and, in consequence, such pipe is more subject to leakage, to overcome which a comparatively larger number of bands should be used. Mr. Rust.

The Great Northern Power Company has four redwood pipes, each about 3 000 ft. long. The first three were constructed about 10 years ago, and evidently an insufficient number of bands were used for this material. The pipes leak quite badly in places, and the bands have to be tightened frequently. This is a more serious matter than would at first appear, as the pipes are covered, and it is expensive to excavate the back-filling in order to tighten the bands. In fact, it is now found that merely tightening the bands will not remedy the trouble, as they sink into the soft wood so that the leakage soon becomes as bad as ever. The fourth pipe, which was built about 3 years ago, had a much larger number of bands, and thus far it has been satisfactory.

It is reported that the failure of the redwood pipe used by the City of Provo was caused by the bands under pressure sinking into the wood to such an extent as to cause very serious leakage. Recently, a number of engineers have used Douglas fir on rather large and important pipe lines, not only because they considered that where the wood was kept saturated, Douglas fir would last as long as redwood, but also because it is stronger, will stand more distortion without leakage, and is much cheaper.

The Guanajuato Power Company has also had some trouble with one of its redwood pipes on account of the silt in the water scoring the bottom of the pipe. There has been considerable wear along the bottom staves, particularly at some of the butt joints, where a small pocket has been eaten into the staves, about one-third of the way through. This pipe has been in use for about 10 years.

One of the largest users of wood pipe is the Denver Union Water Company, which has about 100 miles, ranging from 24 to 60 in. in diameter. The Company's first large pipe, laid in 1890, is of long-leaf pine, and is still giving good service. The next pipe, laid a couple of years later, is partly of pine and partly of redwood. Since that time, Douglas fir has been used in all the wood pipe built by the Company, and it has evidently been found very satisfactory.

There are also various examples of pine pipe in the East, which have proved satisfactory, one of which might be mentioned. The D. E. Converse Company, at Glendale, S. C., has a 78-in. pipe made of short-leaf pine, which has been in use for 26 years. There is back-filling around the lower half of the pipe, and the pressure is comparatively light, only a few feet head. Although some decay has taken place on the outside of the staves, the pipe is still in use and still satisfactory.

The author speaks of fir and pine being coarse-grained, but such wood is hardly suitable for pipe, and is not used where the specifications

Mr. Rust. are properly drawn and the material properly inspected. In fact, there is now a specification covering the density of fir, which it is to be hoped will be generally adopted. Fir and pine staves have not been made any thicker than those of redwood, and have been quite satisfactory under high pressure.

A large quantity of kiln-dried fir has been used for pipe, and has proved satisfactory. No doubt kiln drying does make redwood very brittle; it should be air-dried. This, however, is a disadvantage of redwood rather than of fir.

The speaker has no desire to advocate the use of fir in preference to redwood, as he considers the latter equally satisfactory under proper conditions. However, it would be a mistake to create an unfair prejudice in favor of one as against the other.

There has been considerable discussion lately as to whether wood pipe should be buried or left exposed, and as to whether the outside should or should not be painted. There is no doubt that a pipe should be buried completely at least 2 ft. below the surface, or left entirely exposed. However, in the latter case, the necessary supports are a matter for serious consideration, especially for pipes of large diameter on soft ground.

In a dry climate, where a pipe is buried in a dry, sandy, or pervious soil, the ground will absorb the moisture from the outer shell of the wood, so that it cannot be kept saturated, and this causes a condition favorable to decay. Painting the outside of any pipe under such conditions would be most advisable, but if the pipe is laid in wet soil, where the ground itself is always saturated, it does not seem logical to paint the wood. Also, it is advisable to keep the pipe away from all vegetable matter and growing plants, the roots of which will absorb moisture from any wood and cause it to decay.

Creosoting may be advisable to meet some of these special conditions. It is now being used to some extent, and creosoted fir under ordinary conditions costs only a little more than redwood.

As to the statement that "a coating, to be effective, should be applied diligently and often", it has generally been considered necessary to coat a pipe only where it is buried, in which case, of course, it is impossible to paint it often.

Regarding the standard specifications for bands submitted by the author, as stated, it is quite common practice to use a factor of safety of 4 against breaking due to tension caused by the hydrostatic pressure. However, this does not allow anything for the additional tension required to keep the staves tight and prevent leakage; and a factor of safety of 5 should be used, or some additional allowance should be made for the extra tension required for tightness.

A working stress of 800 lb. per sq. in. for the bearing strength of the wood is excessive. It has been found that one of the causes

of decay is the bruising of the wood through excessive pressure on the bands. A working stress of 400 lb. per sq. in. would probably be more satisfactory, and would not require the addition of any large number of bands, except in the cases of pipes of small diameter. It hardly seems reasonable to use a larger working stress in this case than that usually considered good practice for other structures. The experience of the Great Northern Power Company, already mentioned, bears out this contention. Mr. Rust.

Regarding the thickness of the staves in Table 2, there is a jump from 2½ to 3½ in. at a diameter of 6 ft. The Douglas fir manufacturers have an intermediate standard thickness of 3 in., which can be used for pipes from 6 ft. up to 9 or 10 ft. in diameter. This has been quite satisfactory, and makes considerable saving in the quantity of wood required for pipes between these sizes. There seems to be no reason for not using an intermediate standard thickness of 3 in. with redwood.

Regarding the author's proposed specifications dividing wood pipe into three classes, *A, B,* and *C,* according to the wood used, apparently the only reason for this is to have an opportunity of putting redwood in a supposedly much superior class by itself, which, of course, would give the manufacturer of redwood pipe a good excuse for asking higher prices for his product.

It would be much more logical to vary the thickness of the staves, the band spacing, and other details which have more effect on the cost of a pipe, and to have three tables such as Table 2, one for each of the different classes. Otherwise, the specifications are those generally proposed by the manufacturers of redwood pipe.

As stated by the author, there is no reason for not making the specifications for fir as strict as those for redwood. However, this is the fault of the lumber mills. The proprietors of Douglas fir mills have not shown themselves to be very broad-minded in many ways, and have done nothing to encourage the use of wood pipe.

It would be of considerable advantage if the wood stave pipe manufacturers had an association, organized on the same lines as the Portland Cement Manufacturers' Association, which would be in a position to give disinterested and authoritative data regarding wood pipe on which engineers could depend; and it would result, no doubt, in the use of a much larger quantity. It would also be of considerable help to engineers if manufacturers would keep a stock of pipe or pipe material on hand at some point in the East, so that in case of emergency it would not be necessary to wait a month or two in order to obtain material from the West.

Wood stave pipe has been in general use in comparatively important works for about 30 years. This should be a sufficient length of time for engineers to judge of its permanence, the precautions to be taken

Mr. Rust. with it under different conditions, and to determine what constitutes a safe design. However, as the author states, many engineers in the East still regard such pipe with suspicion, owing chiefly to reported failures which, when investigated, show either that wood pipe should never have been used, or that proper precautions had not been taken to meet the special conditions. There has been ample experience with such pipe, and much information has been published regarding it. It is now possible to find out what constitutes good practice, and, of late, there have not been as many failures reported as in the early years of its use. At the present prices, wood stave pipe costs only about one-third or one-fifth as much as steel pipe, under conditions where either could be used.

Mr. Robbins. F. M. ROBBINS,* ESQ.—It has been suggested that there is a lack of co-operation among the wood stave pipe manufacturers. This is true; and it has held back the development to which good wood pipe is entitled. A closer association would result in the elimination of the poorer pipe which causes most of the unpleasant notoriety. It is not a question of redwood or fir, but of proper design and construction, as investigation shows that most failures in wood pipe are due to design by inexperienced engineers, or construction by outside contractors. A standard specification for wood stave pipe is urgently needed to guide the engineer and eliminate the contractor who builds on a price basis only in keen competition with the established pipe builders.

As stated in the paper, there is "provision for rigid supervision of the bands, though adequate stress is not laid on the requirements for the shoes, which, after all, must be capable of developing the full strength of the band."

The cases cited are only a few of those in which shoes were used which developed only partly the strength of the bands. The cost of the shoe is insignificant as compared with that of the band, and yet its weight, on lines where price competition is keen, is frequently reduced to a point where the value of the steel wasted amounts to several times that of the shoes, and the pipe banding is less than half as efficient.

Furthermore, tons of excess weight have been buried where the shoes were heavier than needed, or where the metal in them was not properly distributed.

Standard lines of properly designed and thoroughly tested shoes have been developed, and these can be used with safety where the specifications are rigid enough to admit no variation or substitution by the contractor.

The specification for pipe shoes advanced by Mr. Partridge is much better than those heretofore offered, but it is not yet as rigid as it

* Secy., The Marion Malleable Iron Works, Marion, Ind.

Mr. Robbins.

should be. The advance in malleable iron has made it possible to secure an even better quality, and the specification should read: "Shall have a minimum tensile strength of 45 000 lb. per sq. in., and a minimum elongation of 7½% measured in 2 in."

The strength of the shoe should be stated definitely as the elastic limit, there having been cases where this was evaded by using the ultimate strength of the shoe as compared to the elastic limit of the band.

The coating of malleable iron is a question that will bear further investigation, as, generally speaking, malleable iron does not corrode, to an appreciable extent, after the first or surface oxidation. The coatings now applied are unnecessary in ordinary circumstances; and, under abnormal conditions, galvanizing or coating with red lead should be recommended.

Mr. Boucher.

WILLIAM J. BOUCHER,* ASSOC. M. AM. SOC. C. E.—The use of wood stave pipe is somewhat limited in the East, but a pipe of this material has been completed recently for the water supply of Watervliet, N. Y., under the direction of G. R. Solomon and P. H. Norcross, Members, Am. Soc. C. E. In New York City, however, there is a type of stave pipe which is hidden from the public view so completely that comparatively few are aware of its existence. Reference is made to the sewer outlet barrel pipes built under the piers on the North and East River fronts. As a sewer outlet at the bulkhead wall would be very objectionable, all sewers are extended to the pier head line by constructing the wood barrel for the full length of the pier at the foot of the street in which the sewer is built. These wooden pipes have been the subject of special study by the Bureau of Sewers of New York City for some time, and are now standardized.

Of course, such pipes need not be designed for the high pressures encountered in water supply systems, as the sewers seldom flow full. They are built of creosoted lumber in order to protect them from the effects of alternate wetting and drying, due to the rise and fall of the tides. The invert is frequently at or near mean low water (in order to obtain as much fall as possible), and this causes the upper portions of the barrel to be submerged only for the short period of high tide. Fig. 1 shows the design for one of the most recent sewers.

The following are the standard specifications of the New York City Bureau of Sewers:

SPECIFICATIONS FOR WOODEN BARREL SEWER.

"Wooden barrel sewers shall be built of creosoted wooden staves held in place by galvanized metal bands, and supported by timber framework.

"All the timber used in the construction of the barrel staves shall be sound commercial short-leaf yellow pine, free from the following

* New York City.

Mr. Boucher.

defects: large, loose, unsound or hollow knots, through shakes or round shakes, worm holes and knot holes. All pieces shall show, either, one heart face or two-thirds heart on both sides.

"Timber required for the permanent support of the sewer shall be long-leaf yellow pine of the quality known as 'Merchantable,' graded in accordance with the rules known as the 'Inter-state Rules of 1905, Adopted by the New York Lumber Trade Association.'

"The barrel staves shall be accurately milled on all sides to exact shape, so that they will form a sewer with tight joints true to the required dimensions.

"The Contractor shall make such tests of the materials and samples as required, and afford every facility requested for measuring tanks, cylinders, gauges, etc., and for taking and analyzing samples as often as may be deemed necessary, including the use of a properly equipped laboratory and other necessary apparatus. The manufacturer shall equip his plant with all necessary gauges, appliances, and facilities to demonstrate that the requirements of the specifications are being fulfilled.

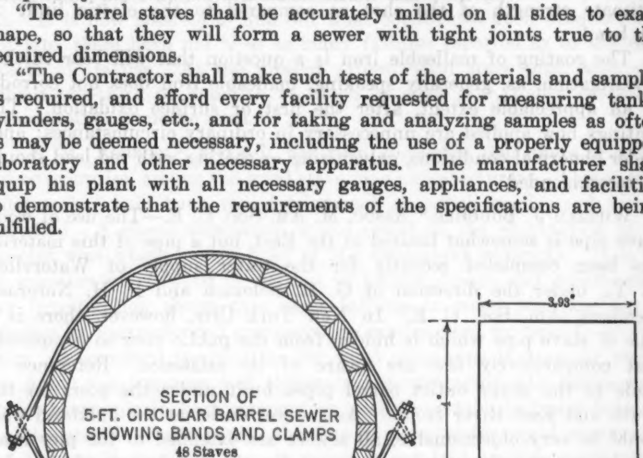

FIG. 1.

"Upon notice from the Contractor that it is ready for inspection, all timber and staves (after the staves have been milled) shall be examined at the one time prior to creosoting. Rejected timber shall be separated from that approved, and the approved timber only shall be creosoted. The Contractor shall give ample notice to the Engineer so that he may arrange for inspecting the creosoting at the place of manufacture.

"The creosoting of the timber shall consist of two operations:

"*a.*—The preliminary application of steam and vacuum.

"*b.*—The injection of a minimum average of 18 lb. of oil into each cubic foot of timber. In addition to this minimum average, additional oil shall be injected into the timber, depending upon the physical condition of the timber, sufficient to render it possible to meet the 5% absorption test specified herein.

"The preservative to be used shall be a distillate of coal gas or coke-oven tar, and shall be free from all adulteration and contain no raw

tar, filtered or unfiltered tars, or pitches, petroleum compound, or other tar products. Mr. Boucher.

"It shall be completely liquid at 38° cent. and shall have a specific gravity at that temperature of not less than 1.03 nor more than 1.08.

"It shall contain not more than 2% of matter insoluble by hot extraction with benzol and chloroform.

"On distillation, which shall be made according to the Borough President's standard method of tests, the description of which is on file in the office of the Engineer, the distillate, based on water free from oil, shall be within the following limits:

"At 210° cent., not more than 5 per cent.
At 235° cent., not more than 38 per cent.
At 315° cent., not more than 85 per cent.

"The distillate between 210° cent. and 235° cent., shall yield solids on cooling to 15° cent. The preservative shall contain not more than 3% of water.

"Samples of the preservative taken from the treating tank during treatment shall at no time show an accumulation of more than 2% of saw-dust, dirt, or other foreign matter. Due allowance shall be made for such accumulation of foreign matter by injecting an additional quantity into the blocks.

"Before being treated, the timber shall be air-dried to such an extent that its weight per cubic foot does not exceed 50 lb.

"Live steam shall be admitted into the cylinder, and applied to the timber, and gradually raised during a period of 1 hour, to a boiler gauge pressure of 15 lb. at 185° Fahr., and maintained for a period of 2 hours for the timbers weighing 40 lb. per cu. ft.; or it shall be gradually raised during a period of 2 hours to a boiler gauge pressure of 25 lb. at 220° Fahr., and maintained for 5 hours for the timbers weighing 50 lb. per cu. ft.; then a vacuum of about 23 in. shall be applied for 1½ hours for the timbers weighing 40 lb. per cu. ft., and for 2½ hours for the timbers weighing 50 lb. per cu. ft., with the temperature of the cylinder maintained at 150° Fahr. For intermediate weights of timber the above treatment shall be varied as directed. Timbers whose average weight per cubic foot varies by more than 5 lb. shall not be treated in the same charge.

"Oil at not less than 180°, nor more than 190°, shall then be admitted into the cylinder, and the pressure gradually raised during a period of 3 hours to 165 lb., or until an average minimum of at least 18 lb. of oil per cu. ft. has been forced into the timber, and, if necessary, an amount in addition thereto sufficient to render it possible to meet the 5% absorption test specified herein. During this period the temperature of the oil shall not be allowed to fall below 165° Fahr. The free oil shall then be expelled from the cylinder.

"After treatment, the timber shall be held in the cylinder for about 1 hour, and shall then be withdrawn. The treated timber shall be protected from the direct rays of the sun, and shall be loaded for delivery within 48 hours after withdrawal from the cylinder.

"After delivery, but before placing in the structure, the timber shall show such water-proof qualities that, after being dried in an

Mr. Boucher. oven at a temperature of 100° Fahr., for a period of 24 hours, weighed, and then immersed in clean water for a period of 24 hours, and again weighed, the gain in weight shall not exceed 5 per cent.

"The barrel sewer shall be built so as to form a continuous structure, unless otherwise directed. If built as a continuous structure, each stave shall be placed so as to form a lap of at least 4 ft. with the adjoining stave. Each stave, as put in place, shall be nailed to the adjoining stave, in place, with tinned-wire nails 7 in. long, spaced at intervals not exceeding 3 ft. Staves shall be thoroughly fastened to each other and to the frame and chocks, and shall be not less than 20 ft. in length, except at closures.

"All joining, framing, and mortising shall be done in a workmanlike manner. Holes, of the sizes required, shall be bored for all spikes and bolts.

"The acceptance of material at the plant of the manufacturer is tentative only, and the City reserves the right to reject shipments, in whole or in part, after delivery on the line of work, if the material fails to comply with the requirements of the plans and specifications.

"Galvanized wrought-iron or steel bands, saddles, hangers, manhole covers and frames, angles, bolts, washers, etc., including all metal required in the permanent structure, shall be furnished, placed, and adjusted as shown on the plans and in accordance with the requirements of these specifications.

"The steel and wrought iron shall comply with the requirements, and be subject to the tests provided in the standard specifications most recently adopted by the American Society for Testing Materials.

"The bands, saddles, hangers, manhole covers and frames, shall be brought to true dimension and shape indicated on the plans before galvanizing. The extra thickness of the band shall be obtained by 'upsetting' the ends. Allowance shall be made in threaded articles so that the parts may screw together after galvanizing without recutting the threads. Samples of the metal to be used in the structure shall be submitted for test and approval before galvanizing. Approved metal shall be galvanized as follows: the iron and steel surfaces must first be cleaned of all scale and rust by means of steel brushes and a dilute sulphuric acid bath. When cleaned, the metal shall be plunged into a bath of molten zinc covered with sal-ammoniac.

"The galvanizing must show an even distribution of zinc over the entire surface of the steel or iron, bright in color, and not blotchy in appearance. At least 1 lb. of zinc shall be applied to every 6 sq. ft. of surface."

Mr. Ralston. J. C. Ralston,* M. Am. Soc. C. E. (by letter).—A more or less continuous experience of nearly 18 years in the use, design, and operation of wood stave pipe, and a fairly intimate knowledge of manufacturing methods, during which time the writer has had to do, in a consulting capacity, with more than 3 840 miles of all sizes, including more than 40 miles of continuous stave pipe of an average diameter of 70 in., impels him to discuss briefly certain phases of

* Spokane, Wash.

Mr. Partridge's paper, and to call attention to some incorrect statements and immature conclusions. Mr. Ralston.

The author says, on page 442:

"In machine-banded pipe there is absolutely no basis for the present-day so-called specifications."

This needs no comment other than to add that the specifications suggested in the Appendix are not equal to those of the best practice. The basis, foundation, groundwork, or controlling principle on which the author's or any other engineer's specifications for wood pipe are drawn, is the same, namely, the need for a wood pipe.

The evolution of the wood pipe specifications has passed through a long period, and has engaged the efforts of such distinguished engineers as the late J. D. Schuyler, J. T. Fanning, A. L. Adams, Members, Am. Soc. C. E., and others, not to mention any of the present-day lesser lights, and it is known that they found a very substantial and satisfactory basis for both the specifications and the pipe.

Again, in reference to fir (yellow pine, Oregon pine), on page 451, the author states that the oldest lines have been built not more than 10 years.

It is useless to encumber the record with a long list. Two or three citations may be permissible: The Garoga River Project, in Fulton County, New York, is noteworthy. This power plant was built in 1850, and served until a fire destroyed the mill in 1903. The original project comprised a triangular timber crib dam (Fig. 2), about 15 ft. high, much the same as those of the simpler types constructed by Mr. James F. Smith, Chief Engineer, in 1818 and 1819, for the Schuylkill Navigation Company—structures which served from 40 to 50 years. The dam was connected to the mill about 600 ft. down stream by a 30-in. wood stave pipe, through which the water was delivered to an overshot wheel about 10 ft. in diameter. The pipe staves were of hard pine, 2-in. scantling, jack-planed on their edges to the proper bevel, and then assembled. The bands or hoops were of 2-in., 16-gauge, old-process, puddled iron, and were as well preserved as the staves when the line was put out of commission. Angles in the pipe line were turned through wood stave pipe wells, after the manner of a sewer. The line has fallen into decay since the mill was burned in 1903.

Aside from the historical engineering interest of such a well-designed piece of work, as well as its significance as a forerunner of the hydraulic end of the modern power plant, the three outstanding features of interest at present are that the staves were of hard pine, that they were in substantially as good a state of preservation as the iron bands at the end of 50 years, and that the line had a life so much longer than 10 years.

Mr. Ralston.

In 1913, a new power project replaced this primitive plant, and includes about 2 miles of 78-in. wood stave pipe under a maximum head of 160 ft. (Fig. 3.) After an extensive inquiry into the merits of various woods, Douglas fir was selected for this pipe.

Another of the pioneer lines in wood stave pipe construction is that built by the late Mr. Fanning, at Manchester, N. H., in 1874. This was a buried line, and was put in operation in the spring of that year. It continued in operation for 40 years, and was finally displaced, not because the pipe was in bad condition, but because a new power installation with a higher head and a new location had to be chosen. Mr. Fanning's line was 72 in. in diameter, and was of 4-in. "hard pine staves * * * cut from pitch-pine plank."*

Here, again, is another notable example of hard pitch-pine staves serving for four decades instead of one.

Another classic example is the Ogden pipe line. It contains 27 000 ft. of Douglas fir stave pipe, 72 in. in diameter, under a head of from 55 to 117 ft., and was constructed in 1896.

Henry Goldmark, M. Am. Soc. C. E., describing this line, stated, among other things, "A new departure, too, is the use of Douglas fir in place of California redwood. The former timber is much harder and stiffer."† D. C. Henny, M. Am. Soc. C. E., in the ensuing discussion, remarked, "The use of Douglas fir in stave-pipe construction can hardly be called a new departure, as almost all the stave pipes built in Washington, Oregon, and British Columbia were constructed of that material." The late Arthur L. Adams, M. Am. Soc. C. E., in a continuance of the same discussion, added, "The author [Mr. Goldmark] is mistaken in thinking the stave pipe [Ogden line] a pioneer, either in its diameter or in the use of Douglas fir for staves. Several conduits 6 ft. in diameter have previously been built, while the same quality of timber has been repeatedly employed."

Here, then, are prominent engineers engaged in discussing this type of pipe with Douglas fir staves, and not redwood, 20 years ago; and, later, in the U. S. Forest Service *Bulletins,* there are references to Douglas fir or Oregon pine as "the most important of American woods", and "as a structural timber it is not surpassed."

The writer presumes to assert that had the author known more history, and had he been familiar with woods other than redwood, he hardly would have made the following misleading statements:‡

"In all the discussions it is noticeable that there are no references to cases where wood stave pipe has been constructed or operated successfully, or unsuccessfully, for a number of years. Actual cases

* *Transactions,* Am. Soc. C. E., Vol. VI, p. 69.

† *Transactions,* Am. Soc. C. E., Vol. XXXVIII, pp. 268, 311–312.

‡ Pages 435 and 451.

FIG. 2.—DAM AND INTAKE OF GAROGA RIVER PLANT EIGHT YEARS AFTER IT WENT OUT OF COMMISSION.

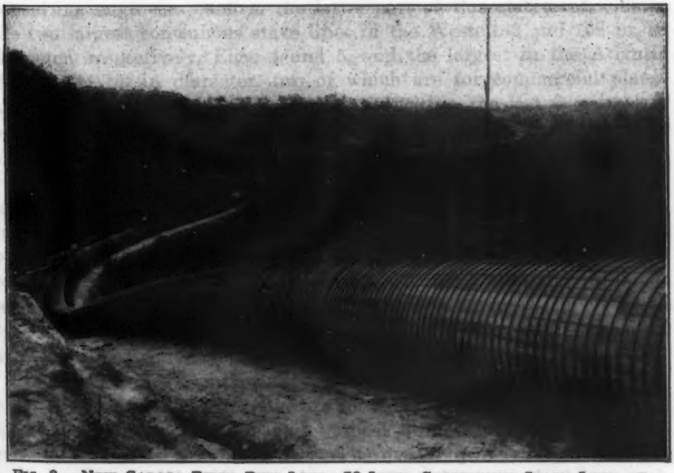

FIG. 3.—NEW GAROGA RIVER PIPE LINE: 78-INCH, CONTINUOUS-STAVE LINE, OF DOUGLAS FIR.

where it has been in service long enough to give an idea of its durability are wanted by engineers." Mr. Ralston.

* * * * *

"Fir is the pipe that has failed, the oldest lines having been built not more than 10 years * * *."

Yet, at the same time, he admits that "Fir and pine are pitchy woods." The inescapable but (the writer is persuaded) unintentional paradox by the author is that pitchy woods are long-lived.

The superior physical and mechanical qualities of Douglas fir are now well known to the Profession, and it is unjust, misleading, and not in the interest of scientific truth for the author to state directly and by repeated implications that Douglas fir is inferior to redwood. Both are excellent woods, the best known for stave material. Although the writer's experience of 18 years in stave pipe construction definitely commits him to Douglas fir, he would not belittle the excellent qualities of redwood. If he did, he could cite examples wherein redwood lines have been repaired with Douglas fir staves. Doubtless, instances might also be cited involving the opposite practice.

Side by side and under identical conditions, one wood will last as long as the other, except in the case of continuous stave lines, when Douglas fir will take the premier position. Not only is this the writer's mature judgment, but the latest practice of some of the leading engineering corporations, and some of the most eminent consulting engineers confirm the latter part of this statement. Thus, the two largest continuous stave lines in the West, 162 and 168 in. in diameter, respectively, Figs. 4 and 5, and the largest in the Atlantic States, 144 in. in diameter, two of which are for commercial plants and one for railroad electrification, built within the last 3 years, are of Douglas fir. One of the Western lines is buried, the others are above ground.

In the matter of the wire-wound type, from 2 to 24 in. in diameter, used in domestic water supplies, the greatest annoyance has arisen in cases where the purchaser has insisted on the cheapest possible pipe, with sap staves and light wire, regardless of the assurance that heavier wire and selected, sapless staves would give better and longer service. In the course of time—the purchaser having been displaced by a new operator and the purchaser's demands forgotten—criticism, and sometimes condemnation, of all wood pipe has resulted.

It is true, as the author suggests, that the nature of the back-fill often has much to do with the life of the pipe. Careless or ignorant methods in laying the pipe and in back-filling the trenches are the cause of trouble and reduced life. The most important consideration, next to the quality of the pipe, is the selection of clean earth

Mr. Ralston. back-fill devoid of vegetable mould, roots, grass, sod, or other unstable material, as well as its proper compaction, preferably for the full depth of the trench. Percolating swamp waters are very destructive. Their carbon dioxide is the arch enemy of wire, rods, and steel pipe, although not particularly harmful to the stave wood. The life of a buried pipe is always increased materially when percolating waters, free or entrained oxygen, leaching processes, or the formation of alkaline salts are excluded. The writer has examined municipal pipe lines which had been laid for 10, 12, and 14 years under compacted clay back-fill, and found the galvanizing on the wire as bright as when it left the factory; and the tar coating on the stave had to be scraped away, in order to reach the wood, which was in perfect condition. An indefinite life could almost be ascribed to such lines.

The writer can well understand how such careful methods of back-filling might largely account for the well-preserved specimens of wood pipe that have been exhumed, from time to time, in London and some of the older American cities, which had been buried from one to two centuries.

On the other hand, the writer has examined lines, the porous back-fill of which had been percolated and partly washed away by continuously flowing swamp water, and has found that the wire banding had been destroyed in 3 years. Thus, there are other elements than the selection of good materials, or the manufacturer's technology, which profoundly affect the life of wood stave pipe.

The use of wood stave pipe and the science of its construction, when left in the hands of experienced engineers, now rests on a sound, intelligent, and definite economic basis. Its use is no longer experimental or a makeshift. Neither are its design, materials, manufacture, or erection invested with the uncertainties ascribed by the author. It is true that in wood pipe, as in nearly all classes of engineering products, poor materials and workmanship are sometimes found, but that does not condemn the type, nor is the technology of the subject so impoverished that it can be metamorphosed by the one-sided specifications suggested by the author.

Except in the most obvious cases of temporary service, there should be only one standard of excellence, namely, the best. The variables in dimensions, strength, and materials are fixed by service demands, and are well understood and simple.

Contrary to the author's statements, the quality and kind of material in the staves are well defined in the manufacturers' and in the latest Reclamation Service specifications. The following is quoted from Specifications No. 73-D, U. S. Reclamation Service, Shoshone Project, Wyoming, dated March 26th, 1917, and may be taken as representative of good practice:

Fig. 4.—Fourteen-Foot Line of Montana Power Company, Near Great Falls, Mont., of Douglas Fir.

Fig. 5.—Another View of 14-Foot Line of Montana Power Company.

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"All lumber used in staves shall be Douglas fir. It shall be sound, straight-grained, and free from sap, dry rot, checks, wind shakes, wane, and other imperfections that may impair its strength or durability. Pitch seams will be permitted in not more than 10% of the total number of pieces, it showing on the edge only, and if not longer than 4 in. nor wider than $\frac{1}{16}$ in.; no through knots or knots at edge or within 6 in. of ends of staves will be allowed; sound knots not exceeding $\frac{1}{2}$ in. in diameter, not falling within the above limitations, not exceeding three in number within a 10-ft. length, will be accepted. All lumber used shall be seasoned by not less than 60 days air drying in open piles before milling, or by thorough kiln drying. All staves shall have smooth planed surfaces, and the inside and outside faces shall be accurately milled to the required circular arcs." Mr. Ralston.

The foregoing fixes the quality and kind of material. The thickness of the finished stave will depend on the diameter of the pipe and the pressure under which it serves. Stave thickness varies from 1 in. in the smallest machine-banded pipe to 4 in. in the largest continuous stave pipe.

The paper and suggested specifications ineptly seek to give the impression that all first-class stave pipe should be made only from clear, one-year, air-dried redwood, and that only the inferior grades of pipe may be made of Douglas fir. This is noticeably conspicuous in the suggested "Specifications for Continuous-Stave Pipe, Class *A*," and the "Specifications for Machine-Banded Pipe, Class *A*." In both these Class *A* types, the writer quite agrees with the author that, where redwood is used, it should be a clear, air-dried product, for the reason, as seems implied, and as is indeed true, that kiln-dried redwood is brittle and structurally inferior. The writer's best judgment and experience, however, are (and a marked preponderance of the best practice throughout the country confirms him) that Douglas fir, either air- or kiln-dried, should also have been specified, unless it was the author's purpose to write exclusively a redwood specification. If, however, he desired to write a standard, Class *A*, wood stave pipe specification, then, in the interests of engineering fact, practice, and non-partisan purpose, Douglas fir should not have been omitted. If redwood is to be substituted for Douglas fir, then, considering equivalent strength, if the safety factor is closely involved, as well as the greater porosity of the former wood, the thickness of the redwood stave should be correspondingly increased.

It is granted that the grain in redwood is not so coarse as in Douglas fir, but the mechanical ultimates of redwood are less than in fir; yet it should be noted that the soft or winter-growth portion of each annual ring in redwood is much more porous than the similar soft portion of each annual ring in Douglas fir. Nevertheless, the

Mr. Ralston. whole question of percolation and saturation in either material is largely academic, if not fanciful. Both woods, or any wood, will become saturated and remain so, as long as the pipes are full of water under a sensible pressure. The best practice now fixes the thickness of the stave as such that, when the requirements of saturation and strength are satisfied, the question of percolation automatically disappears. Any decrease in the standard thickness of stave, either in fir or redwood, should not be permitted.

The writer has never seen nor heard of a case of "excessive percolation" in any pipe, except where the staves were thinner than standard practice. Some redwood machine-banded pipe has been manufactured with unusually thin staves, but it is believed this practice is questionable and should be discouraged.

It is only in the case of intermittent service, such as in irrigation lines, where saturation becomes important. Here, happily, a definite solution of the problem is at hand at a slightly increased cost, that is, to creosote the staves, both fir and redwood, just as the best practice now requires that the wooden sleeves of wire-wound standard pipe shall be creosoted, because of a less perfect saturation in the sleeve than in the pipe.

The writer partly agrees with the author when, on page 451, he says:

"The greater number of failures possibly originated at the joints. The outer edges of the staves in the collars, when wood collars were used, decayed rapidly owing to the fact that fir needs saturation for preservation, and saturation was not secured at those places."

If the author's diction had been more precise, and he had said that the ends of the staves in the collars decayed more rapidly than the walls of the pipe, owing to the fact that all woods need saturation to secure the best preservation, then the writer would agree with him. Redwood does not enjoy a special dispensation of immunity from decay. Metal collars are not a success, and are more or less obsolete. The open-tank creosoted wood collar has proved superior and satisfactory. It should be borne in mind, however, that when the entire pipe is to be creosoted, the process of creosoting must then be with the closed-tank, pressure method, and that the average oil content must be not less than 8 lb. per net cu. ft. of wood. The open-tank or dipping method for collars is used successfully where the pipe proper is not creosoted. The reason is obvious. Clear kiln-dried Douglas fir, when given the open-tank treatment, will get a thorough penetration in the end grain to a substantial depth. The penetration into the side of the stave will vary, depending on the nature of the lumber and the location of the annual rings in relation to the surface of the stave. Collar troubles arise from decay that sets in on the ends of the staves and works back, finally undermining the

wires. If the ends of the staves in the collars are protected by open-tank treatment, from 90 to 95% of the protection afforded by pressure treatment is secured at a greatly reduced cost. Pressure creosoting, fortunately, permits the use of sapwood in staves, and where such method is used, the specifications should be correspondingly changed. The writer believes that properly creosoted wood pipe for high-class permanent work will meet with great favor, and has an encouraging future. Mr. Ralston.

An instructive series of tests was recently made by the Engineering Department of the West Coast Lumbermen's Association, in co-operation with the University of Washington, on twenty-three Douglas fir staves, taken at random, which had been in continuous service for 16 years on the Cedar River pipe line of the City of Seattle. On the sections of pipe examined, and from which the staves were taken, "there was not a single stave in either pipe section which showed any signs of decay." The specimens were tested in compression parallel to the grain to determine their crushing strength. The staves had been under 22 lb. hydraulic pressure for that length of time. They gave an average dry weight of 30.8 lb., and a maximum average crushing strength of 3 870 lb. per sq. in. The size of the average specimen taken from each stave was 1.63 by 5.55 in. The average number of rings per inch was 18.

Douglas fir, having a weight of 30.8 lb. per cu. ft., may be expected to have a crushing strength of approximately 4 400 lb. per sq. in.* If the original strength of the staves fulfilled this expectation, then, after 16 years of continuous saturation at a pressure of 50 ft., they exhibited 88% of the strength of new fir. Inasmuch as the mechanical ultimates of Douglas fir, with the usual safety factor of 4, are never reached, except when staves are made thinner than required by standard practice, this 12% reduction in strength is of no consequence in fir stave stock.

In the matter of curing the stave stock, extensive tests, as well as experience, have shown that the resultant difference between air-dried and properly kiln-dried Douglas fir staves is altogether negligible. In fact, there are some features of kiln-drying more favorable than air-drying, notably among others, is the tendency which this process has to develop the full extent of pitch pockets more visibly than by air-drying, whereby such defects are more easily detected by the inspector, thus making the culling process more certain. At least one of the leading Pacific Coast pipe factories has developed a special kiln method which seems to give excellent results. The ordinary dry-kilns of the lumber companies, and the methods of kilning by such companies are not suitable to the proper curing of pipe staves. Special kilns and their proper operation are the sole

* *Bulletin* 88, U. S. Forest Service, p. 26.

Mr. Ralston. factors in the curing of Douglas fir stave stock, in order to make it as good as if air-cured.

It is proper, and no doubt customary, to give all wire-wound pipe, both fir and redwood, a bath in hot tar or asphalt before it leaves the factory, and, while the coating is still hot, to roll the pipe over a bed of saw-dust. This treatment is a preservative, and adds to the life of any pipe; but, even if it had no preservative quality, its value as a protection against abrasions, rough handling, and the wear and tear incident to transit, is a wise and time-honored practice. The author's contention that such treatment "increases the weight materially" is misleading, as it does not generally affect the shipping charges. The fact is that, in most cases of the standard freight car, the total weight of a load of pipe is less than the minimum car-load weight prescribed by the railroads.

Although continuous stave pipe is a comparatively simple structure, it is a mistake to assume that it can be assembled or laid without the use of expert and experienced supervision, preferably by the manufacturer's engineer, if he is an experienced man. George L. Watson, Assoc. M. Am. Soc. C. E., concludes an excellent article* with a warning to all general contractors to keep hands off, and leave all wood pipe to a regular pipe contractor. The writer is familiar with several large jobs, partly completed by local or general contractors, which had to be pulled down and relaid by an expert.

On page 439, we read: "Fir and pine are pitchy woods, and it is impossible to obtain commercial run lumber without sap, pitch, pitch seams, pitch pockets, and knots." The Shoshone Project specification, which the writer has quoted in this discussion as being representative, and which he recommends, does not call for "commercial run lumber," but for a stock of definite and select quality. He has never known of commercial run lumber, either fir or redwood, being used for first-class pipe.

That part of the author's suggested specification for Class *A*, Continuous-Stave Pipe, relating to redwood stave stock is not specific enough, and allows too much latitude in the character of the material. Redwood, like fir, should be straight-grained, devoid of checks and wanes, and should have no knots at the edges of the staves, nor within 6 in. of the ends, nor any defects which may impair its strength or durability. (The expression, "materially impair" should not be used.) Moreover, the width of the metallic tongue should be 2 in., because of the alleged tendency of redwood to shrink endwise.

Finally, the basis of classification, given on page 457, in terms of longevity, seems to be more pedantic than empiric, and, therefore, should have small place at this time.

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* *Journal*, Am. Soc. of Eng. Contractors, Vol. IV, No. 7, September, 1912.

O. P. M. Goss,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with considerable interest. It contains some very interesting suggestions, and is believed to be a step in the right direction, at least in so far as it suggests the standardizing of specifications covering the design and construction of wood stave pipe. This subject, however, is very important, and should receive thorough consideration before any definite specification is approved. Mr. Goss.

There are, for example, certain statements in the paper which are not based on the natural laws underlying the most approved use of wood. In offering the following comments for consideration, the writer has endeavored to set forth certain facts which cannot be ignored in the general discussion of wood stave pipe.

On page 439 the author states:

"Fir and pine are pitchy woods, and it is impossible to obtain commercial run lumber without sap, pitch, pitch seams, pitch pockets, and knots. Under conditions of partial saturation, this lumber will not last, and, even with saturation, the pitch and sap will be the cause of deterioration. Most failures are attributable to this fact. There are conditions under which fir or pine will have a long life and give perfect satisfaction."

Pitch does not in any way cause the deterioration of timber. Tests of recent date, made at the U. S. Forest Service Laboratory, indicate that wood containing resin deteriorates a little less rapidly, on the average, than that which is free from this substance. The following quotation is taken from a recent report issued from the U. S. Forest Products Laboratory, at Madison, Wis.:

"*Relation of Resin to Strength and Durability.*—Data on the effect of resin on durability were worked up for 105 samples of long-leaf pine. The results, when considered as averages for four durability classes, indicate that increasing amounts of resin tend to be directly correlated with increased durability. Individual blocks do not necessarily bear out this relation, showing that there are other factors involved."

The following quotation is from page 441 of the paper:

"The cases of redwood pipe already cited illustrate its adaptability, whether laid on the surface of the ground, partly or completely buried, or run through salt marshes or tropical swamps in direct contact with the soil humus. Direct exposure to the rays of the desert sun, and alternate wetting and drying when the pipe is used intermittently in irrigation systems, do not lessen its efficiency."

The writer cannot see how any one can make such a broad statement, and particularly that "alternate wetting and drying when the pipe is used intermittently in irrigation systems" does not lessen its efficiency. Again, tests made recently at the U. S. Forest Products

* Seattle, Wash.

Mr. Goss. Laboratory indicate that all woods are subject to decay under adverse conditions. It could only be considered good engineering when conditions are prevented which would tend to make any wood less durable. Heartwood from the California big trees, at the end of a 12-month test, showed a loss in weight of 35.1%, due to deterioration.* This wood is not the same as redwood, but is similar, and is used here in the absence of similar data for redwood. Western red cedar, an unusually durable wood, under similar conditions, showed a loss of 21.3%, but it is reported that the samples were too wet to give a fair test, which indicates that 21.3% loss is lower than a fair test would have shown. Port Orford cedar, readily conceded to be one of the most durable woods, showed a loss of 22.6%, which, again, is lower than the value would have been had the specimens been less saturated. Douglas fir showed a loss of 28.1%, according to these tests. Deterioration, in this case, also, was somewhat retarded by an excess of moisture. These results show that the most durable woods are likely to decay if subjected to adverse conditions. Due to this fact, no wood pipe, regardless of the species from which the staves are made, should be laid under unfavorable conditions without taking practical precautions against decay of the wood fiber.

On page 444 the author states:

"Fir and pine, being hard woods compared with redwood, and being coarse-grained, having wide rings of hard and soft wood, enter the classification of woods giving excessive percolation, with slow and incomplete penetration. This is caused by the water passing rapidly through the soft summer wood, appearing in drops on the outer surface of the pipe, and of penetrating but slowly, and often through only a fraction of a stave, along the hard winter rings. The result is a stave showing percolation and incomplete penetration at alternate points throughout its cross-section."

Douglas fir, as a matter of fact, is one of the most difficult woods to penetrate with a liquid, and, in this respect, might about as well be classed with metal as with pine. In creosoting timber, throughout the United States, there has seldom if ever been found a wood which has required so much scientific study to secure thoroughly satisfactory impregnation as has been the case with Douglas fir. In the treatment of ties of this wood, it is highly desirable to perforate the sides of each tie with fine holes, uniformly spaced, in order to get an effective injection of creosote oil.

It is usually specified that pipe staves of Douglas fir shall be practically free from all defects, which means that this stock must not be cut from the center of the log, which usually contains most of the knots and other defects. Due to this fact, the staves are cut

* Table 7, p. 90, "Laboratory Tests on the Durability of American Woods," by C. J. Humphrey, of the U. S. Forest Products Laboratory.

from the fine-grained material found on the outer portion of the large fir logs, and not from the coarser-grained material, which is almost always confined to the center portions of the tree. Mr. Goss.

In the selection of pipe staves, care is taken to eliminate coarse-grained material. No difficulty whatever is experienced in eliminating practically all the sap wood, and, in pipe properly manufactured, the sap is never allowed to occur on the outer portion of the stave. Sap is not considered a defect on the inner portion, in a line which is in continuous service, because of the fact that, under this condition, it is always thoroughly saturated.

In Douglas fir staves of medium and fine growth, the summer and spring wood bands of the annual ring are so close together that if either is thoroughly saturated the adjacent one must also be wet.

The soft portion of the annual ring of redwood is more porous than the corresponding part of Douglas fir, as shown by Figs. 6 and 7. Redwood holds a natural moisture content of about 80% and the normal moisture content of Douglas fir is 33%, based on the dry weight of the wood in both cases. These facts indicate a greater porosity in redwood than in Douglas fir. As a matter of practice, however, neither of these woods is justly subject to criticism from the standpoint of excessive percolation.

The author refers to the "soft summer wood and hard winter rings." Technically, the summer wood is the hard part of the annual ring, and is formed during the dry period of the tree's growth; that is, through the late summer and early fall. The soft or porous portion of the annual ring is the spring wood, and is formed in the early spring and summer, when moisture in the soil is abundant and the growing rate of the tree is most rapid.

On page 445 the author states:

"The lumber, therefore, should be perfectly dry before being used. It should be dried by the natural or air-drying process, not by the forced or kiln-drying process. By air-drying only is perfect, sound, strong lumber obtained. Kiln-drying makes brittle and lifeless lumber. Air-drying requires time, and, as lumber should be seasoned for at least a year for the best construction, a large stock of it should be available at all times."

As a matter of fact, better results may be secured by correct methods of kiln-drying Douglas fir lumber than by air-seasoning it. Like any other process of manufacture, kiln-drying has its successes and failures. With a fundamental knowledge of wood and of the law governing successful kiln-drying, entirely satisfactory results are now obtained. It is possible to kiln-dry Douglas fir staves in such a way as to leave them in perfect condition for use. This kiln-drying may be, and is, done to-day so as to produce faultless lumber with its full

Mr. Goss. original strength, in fact, much more than its original green strength, as it comes from the kiln.

Small specimens of long-leaf pine,* ¾ by ¾ in. in cross-section, air-dried for 98 days and re-soaked in water for 47 days, showed a crushing strength of 2 213 lb. The same material (matched pieces) after being kiln-dried 35 days and re-soaked in water 63 days exhibited 2 268 lb. Air-dried, re-soaked specimens of red spruce, handled in exactly the same way, showed a strength of 1 553 lb., and for the kiln-dried, re-soaked, 1 606 lb. Chestnut was also used in this test, and exhibited 1 482 lb. for the air-dried, re-soaked, and 1 573 lb. for the kiln-dried, re-soaked pieces. These figures show that proper kiln-drying does not injure the strength of the wood, as compared to air-seasoning; and it must be remembered that these tests were made on specimens which were water-soaked after being kiln-dried and air-dried, which makes the tests particularly applicable to pipe staves in service.

Mr. H. D. Tiemann, of the U. S. Forest Products Laboratory, one of the best versed men in the United States on the theory of kiln-drying lumber, states:†

"While air-drying is undoubtedly the safest method, the process is ordinarily so slow, requiring a year or longer, according to species and size, that forced 'artificial' drying becomes a business necessity. Moreover, air-drying is by no means always to be preferred to kiln-drying from the standpoint of the quality of the product."

The writer has made tests on Douglas fir and western hemlock, similar to those quoted from *Circular No. 108* of the U. S. Forest Service, showing that kiln-drying operations do not affect adversely the strength of either of these species, as compared to air-seasoned material. This may not be true in the case of redwood or cedar, as the structure of these woods is very different, and the cells appear to collapse under the application of heat very much more readily than with Douglas fir or pine.

On page 446 the author states: "For correct pipe design, only air-dried lumber should be specified." This seems to be so far out of line with good practice that it needs little comment. Practically all the Douglas fir lumber which is put into pipe is thoroughly kiln-dried before use, and any one familiar with this material knows that, under conditions which are favorable to any wood pipe, it gives the best of service.

The author discusses machine-banded pipe, and refers to the difficulty experienced with the banded couplings, due to their lack of durability. He suggests that fir pipe couplings be discarded and replaced by couplings of redwood. This is not the best solution of

* *Circular No. 108,* Forest Service, U. S. Dept. of Agriculture, Table 5.

† *Bulletin No. 509,* U. S. Dept. of Agriculture, p. 2.

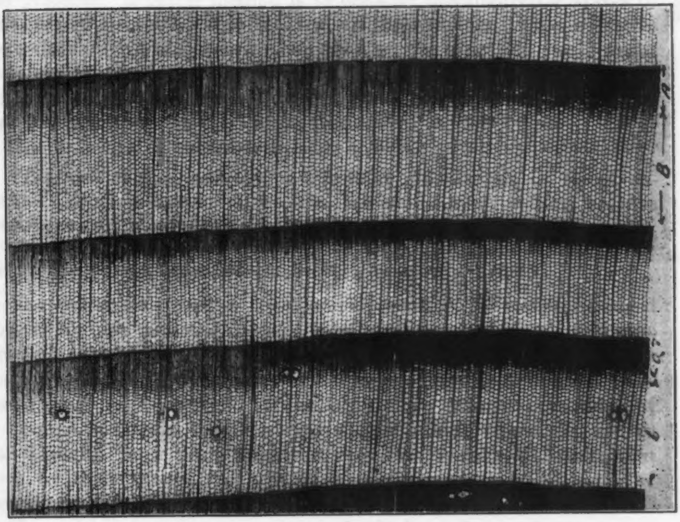


Fig. 6.—Cross-Section, Douglas Fir, Showing Cell Structure. (20 Diameters.)
***A.*—Summer Wood. *B.*—Spring Wood.**

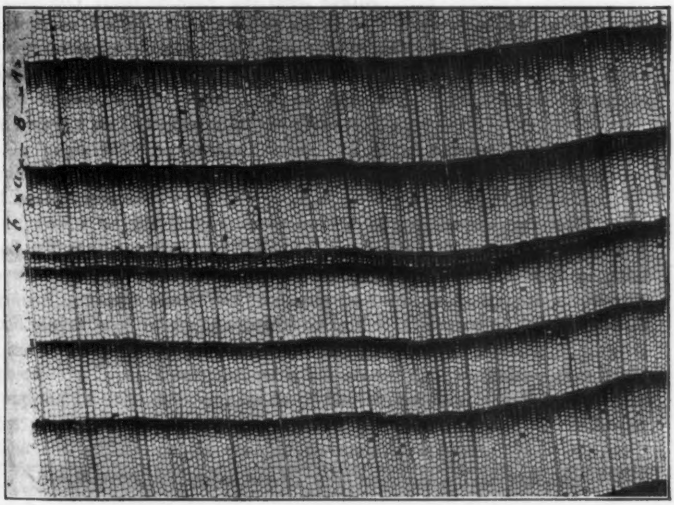


Fig. 7.—Cross-Section, Redwood, Showing Cell Structure. (20 Diameters.)
***A.*—Summer Wood. *B.*—Spring Wood.**

this problem. A better method is to creosote thoroughly the staves of pine, fir, or redwood before making up these collars. This will give them durability even greater than that of the pipe line itself under complete saturation. This is strictly an economical and efficient method, and is to-day being practiced by at least the most progressive pipe manufacturers on the Pacific Coast. Mr. Goss.

The following is quoted from page 451:

"The use of the inserted or slip-joint pipe is not to be recommended. Such a connection weakens the end of every section, because nearly one-half of the shell of the pipe is cut away to make the joint. A reinforcing rod is often used to draw the joint tight, but if the male and female tenons are eccentric, leakage cannot be avoided."

The writer has used a considerable quantity of machine-banded Douglas fir pipe with the inserted couplings, and has found it to give thorough satisfaction. As to the strength of these joints, the writer has had no experience which would indicate that they are not sufficiently reinforced. Redwood staves are usually thinner than those of fir or pine, and this, together with the fact that redwood is weaker than Douglas fir or pine, might account for the lack of satisfaction given by this type of pipe. It is an entire success when made of Douglas fir, in accordance with present standards.

The writer cannot see any reason for leakage due to the tenons or cups being slightly eccentric with regard to the axis of the pipe, so long as they are circular and well manufactured. In his experience, no difficulty from leakage has been found.

Douglas fir has considerable advantage over redwood in strength in compression across the grain. The average strength of these woods* is 570 lb. per sq. in. for Douglas fir and 525 lb. for redwood. These figures show redwood to be approximately 92% as strong in side bearing as Douglas fir.

On page 457 the author states: "For permanent work, redwood should be selected, or fir or pine of high-grade staves kept saturated and well painted."

It would be inviting trouble not to apply the clause "kept saturated and well painted" to redwood, as well as to fir and pine. It is the weak points in a wood pipe that cause the trouble, and in good engineering these weak points should be eliminated by proper means. Wood staves used under low heads or under intermittent service should be creosoted by the pressure process. If, for any reason, this is impossible, a brush treatment with hot creosote, or carbolineum, should be applied to the edges and ends of the staves, and also to the entire outer portion of the pipe line. This treatment should be followed with a hot asphalt or tar coating thoroughly applied.

* *Bulletin No. 108*, Forest Service, U. S. Dept. Agriculture, Table 12.

Mr. Goss.

On page 446, the following is found:

"In regard to the protection of the staves by applications of coatings of paint or disinfectant, little of value can be cited. Many claims are made for the benefits derived from various coatings, but sufficient data are yet lacking for reliable conclusions. It is certain that such protection increases the life of fir, pine, or other woods containing sap and pitch, but its merits on a redwood pipe have not yet been proved.

* * * * * * *

"A coating of at least $\frac{1}{16}$ in. should be the result of the first painting, and repeated examination should be made of the line, and the pipe painted every year or so."

It is safe to say that a protective coating or preservative properly applied will increase the life of any wood pipe. Mr. Partridge states that in fir and pine pipe it is necessary to apply a preservative coating "every year or so." The writer knows of no Douglas fir or pine pipe lines which have been coated as often as this. In fact, it would be impracticable and entirely unnecessary to paint a wood pipe line "every year or so" when such line is buried in the soil. If the proper material is used, the protective coating should remain on the pipe at least 15 years under such conditions. The writer has seen Douglas fir stave pipe uncovered 24 years after it had been laid, and the asphaltic coating was still in good condition. If the pipe is used above ground, it would not be necessary to paint it oftener than once in 5 years, because the natural conditions would tend to retard decay.

Untreated wood stave pipe has demonstrated its value when used with a thorough knowledge of the underlying principle controlling the efficient use of wood. There is in store for such pipe, however, a development which is now making its appearance and will materially change present practice, and that is the creosoting of the staves under pressure as a means of eliminating decay.

Considerable effort has been made by the Engineering Department of the West Coast Lumbermen's Association, particularly during the last 3 years, to improve the methods of creosoting Douglas fir in its various forms. One of the forms already studied is staves. In the methods of treatment which have been applied in the past (the old boiling or steaming processes), previous to the recent developments in the art of creosoting Douglas fir, there has been considerable loss in strength in compression perpendicular to the grain, due to the creosote treatment. This loss, of course, was not desirable in the case of staves, which depend on the strength of the wood in side bearing to resist the water pressure. There was a loss in strength of 30% or more in the staves as a result of either of these treatments. However, by the

process now in use—the "boiling under a vacuum method"—which greatly reduces the temperature of the oil during treatment, or simply treating under low temperatures, there has been no loss in the strength of the staves. Mr. Goss.

The West Coast Lumbermen's Association recently completed some strength tests on staves which had been creosoted by this mild-temperature method of treatment. Twelve Douglas fir staves were selected, which were entirely free from sap wood, and twelve other staves were chosen as nearly like the first group as possible, except that they contained various quantities of sap wood. All these staves were 6 ft. long, and were kiln-dried. A section 1 ft. long was cut off the end of each stave and retained for a control test. The remaining portions of each of the twenty-four staves were treated in the following manner:

They were warmed in creosote oil for 4 hours at 170° Fahr., and pressed from 0 to approximately 100 lb. per sq. in., until they had received about 16 lb. of oil per cu. ft.

The oil was then heated from 170 to 230° Fahr. in 3 hours, and held at this latter temperature for 1 hour. The staves were then removed.

The final heating bath was for the purpose of removing the surplus oil and cleaning the stave.

TABLE 4.—BAND BEARING TEST ON DOUGLAS FIR STAVES, NATURAL AND CREOSOTED. STAVES SOAKED IN WATER ONE MONTH BEFORE TEST.

Condition of stave.	LOAD, IN POUNDS, REQUIRED TO PRESS A SECTION, 3.35 IN. LONG, OF BANDS OF VARIOUS DIAMETER, INTO THE STAVE TO A BEARING OF 60 AND 90° OF ARC.								Average for all tests.
	0.193 in. Diameter.		0.226 in. Diameter.		0.497 in. Diameter.		0.625 in. Diameter.		
	60°	90°	60°	90°	60°	90°	60°	90°	
STAVES CONTAINING SAPWOOD.									
Natural	166	371	218	453	507	1 096	608	1 334	594
Treated	171	414	250	518	620	1 338	756	1 663	716
Treated, in percentage of natural	103.1	111.6	114.7	114.4	122.3	122.1	124.4	124.7	120.5
STAVES ALL HEARTWOOD.									
Natural	196	460	267	574	592	1 291	714	1 572	708
Treated	210	531	332	718	752	1 596	880	1 929	868
Treated, in percentage of natural	107.1	115.5	124.4	124.2	127.0	123.6	123.2	122.7	122.6

Mr. Goss.

After this treatment, the staves were free from excess of oil and easy to handle. They were then placed in a water tank with the untreated pieces, and all were soaked for about 30 days. Then both the natural and creosoted staves were subjected to a band pressure test. In making this test, four sizes of bands were used, as shown in Table 4. Each band was 3.35 in. long, and was pressed into the stave, for the entire length of the band, in a direction perpendicular to the grain of the wood, until it was embedded over an area equal to 60 and 90° of the arc. Figs. 8 and 9 illustrate the method of making the test. The loads required to cause this depth of compression are shown in Table 4. Each stave, natural and treated, was subjected to tests with bands of each size. The results show clearly that creosoted staves of all heartwood as well as those of mixed heartwood and sap wood have strength values even greater than those obtained from the test of natural wood. The results are particularly significant, as the test approaches about as closely as possible the actual condition of staves in a pipe line in service. The creosoted staves uniformly show a slightly higher strength than the untreated staves tested under the same conditions. These results demonstrate clearly the fact that it is possible to creosote Douglas fir staves and retain all their original strength. They also indicate clearly the practicability of permitting sap wood in staves which are to be creosoted. In the staves tested, the penetration of the creosote oil in the sap wood was very complete in every case.

Recent experimental work conducted by the writer has also shown the practicability of perforating the outer surface of Douglas fir staves with small holes systematically spaced, as shown in Fig. 10. The resistance of the oil passing into the wood along the grain is practically nil compared to that found in forcing the oil in across the grain. The fine holes, through which the depth of penetration and the distribution of oil is controlled, are so small and spaced so regularly that they do not reduce the strength of the wood when tested in compression perpendicular to the grain. The result of these perforations is that the depth of penetration and the distribution of the oil can be thoroughly controlled—two very important factors in securing the greatest efficiency from creosoted timber.

Mr. Partridge does not discuss at any length the possibility of the entire elimination of decay in wood stave pipe by an efficient pressure treatment of the staves with coal-tar creosote. This subject should be thoroughly investigated at once, as the proper use of creosote promises to extend greatly the field of usefulness of such pipe, and give it a reliability never before possessed. The writer is fully convinced of this, after having studied the subject very carefully. A large number of test results are available, and more should be obtained.

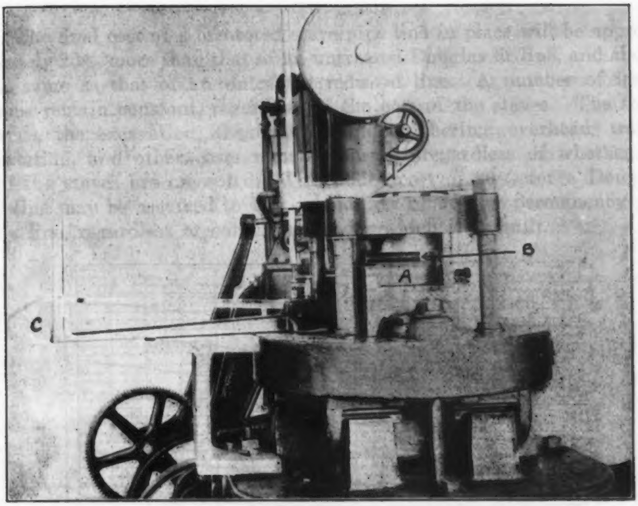


FIG. 8.—METHOD OF MAKING BAND PRESSURE TEST.
A.—STAVE UNDER TEST.
B.—BAND BEING PRESSED INTO STAVE.
C.—INSTRUMENT FOR READING COMPRESSION.

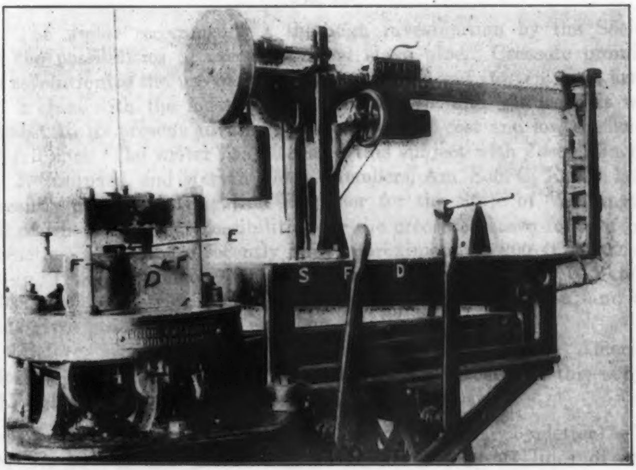
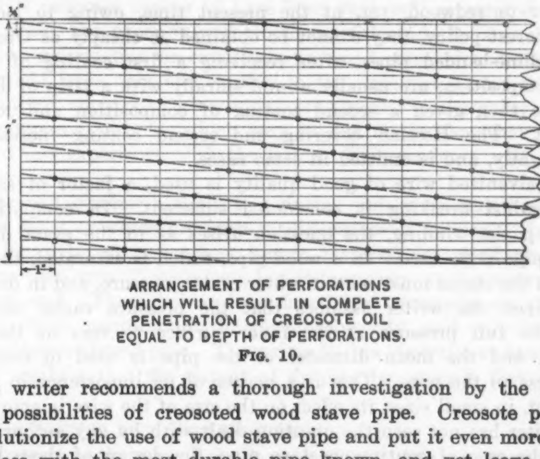


FIG. 9.—METHOD OF MAKING BAND PRESSURE TEST.
D.—CAST-IRON SECTION FOR HOLDING SPECIMEN.
E.—BAND BEING PRESSED INTO STAVE.
F.—SET-SCREWS FOR TIGHTENING STAVE IN POSITION FOR TEST.

Fig. 4.—[illegible]
[illegible]
B—Cast Steel Flasks [illegible]
[illegible]

Fig. 5.—[illegible]
D—Cast Iron Flasks for Machine Moulding.
E—Cast Steel Flasks [illegible]
F—[illegible] for [illegible] Flask [illegible]

The final cost of a creosoted stave pipe line in place will be approximately 20% more than that of an untreated Douglas fir line, and about the same as that of an untreated redwood line. A number of large items remain constant, regardless of the cost of the staves. The steel bands, the excavation, the back-filling, engineering, overhead, transportation, and other costs remain constant regardless of whether or not the staves are creosoted. This added cost of 20% for a Douglas fir line may be assumed to buy a guaranty of greater permanency for any line, regardless of conditions under which it is built. Mr. Goss.



ARRANGEMENT OF PERFORATIONS
WHICH WILL RESULT IN COMPLETE
PENETRATION OF CREOSOTE OIL
EQUAL TO DEPTH OF PERFORATIONS.

FIG. 10.

The writer recommends a thorough investigation by the Society of the possibilities of creosoted wood stave pipe. Creosote promises to revolutionize the use of wood stave pipe and put it even more firmly in a class with the most durable pipe known, and yet leave it with almost all its present advantages as to low first cost and low coefficient of friction. The writer has discussed this subject with Joseph Jacobs, L. J. Stannard, and Marvin Chase, Members, Am. Soc. C. E., the latter recently appointed Irrigation Engineer for the State of Washington, all of whom see great possibilities for the creosoted stave form of construction. Mr. Chase recently built approximately 8 200 ft. of 60 and 63-in. (inside diameter) creosoted wood pipe for the Wenatchee Reclamation District, in the Wenatchee Valley, Washington, and this line gives promise of most excellent results.

Before any standard specifications are adopted, the American Society of Civil Engineers should thoroughly investigate this subject through a committee capable of going deeply into it.

W. H. R. NIMMO,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer, having had occasion to deal with numerous lines of wood pipe, especially of the machine-banded type, for town water supplies, Mr. Nimmo.

* Hobart, Tasmania.

Mr. Nimmo. has been much interested in this paper. Wood pipes as used in Tasmania, whether of the continuous-stave or machine-banded type, are almost always constructed of fir, known locally as Oregon fir. Although some engineers are fully aware of the superiority of redwood, yet, owing to its greater cost, pipes of this timber cannot be obtained from stock, and it is necessary to have them manufactured especially for each job. In the smaller sizes, up to about 6 in., redwood pipes cannot usually compete with cast iron or reinforced concrete. This State possesses some fine timbers, which may possibly prove superior to redwood, yet, at the present time, owing to lack of a sound forest policy, they cannot be obtained as cheaply as Oregon fir.

Machine-banded pipes, after receiving a first coating of bituminous composition, are usually wound spirally with a strip of Hessian, and are then given a second coating of composition and rolled in sawdust. The Hessian covering and second coating increases the cost slightly, and is omitted in some cases.

If galvanized wire of good quality is used, a factor of safety of four against breaking is considered sufficient. In computing the stress on the winding, the question arises as to the exact diameter of the pipe to be used. In a wood pipe, which is saturated, the joints between the staves must contain water under pressure, and in designing such pipes, the writer assumes that the pressure varies uniformly from the full pressure at the inner surface to zero at the outer surface, and the mean diameter of the pipe is used in computing the stress in the wire. This is a matter of no importance in a large pipe, but, in small sizes, its effect on the size of the wire is appreciable. The writer has not seen the question dealt with by any authority.

In the case of continuous-stave pipe, the design of shoes has not always received sufficient attention. A type of shoe is sometimes used in which the tension of the two ends of the band are not in the same vertical plane, thus adding a horizontal bending stress to the direct tensile stress in the bands near the shoe.

A combination of inserted joint with a collar is now generally used on machine-banded pipes, and has been found to be fairly satisfactory.

The increase in shipping weight due to the coating may not be of great importance. Freight by sea is usually charged by measurement, and special rates by rail can frequently be obtained for complete carloads. The smaller freight and handling charges on wood pipe, as compared with other kinds of pipe, however, are often the determining factors in its selection.

In all specifications for machine-banded pipes, a clause should be inserted requiring each pipe to be clearly branded with letters indicating the job, and figures indicating the head for which it is intended. The writer knows of cases where pipes intended to be laid near the

intake of a line, under a low pressure, have been carelessly laid in places where the pressure was comparatively high, resulting in considerable trouble in maintenance. Mr. Nimmo.

E. A. MORITZ,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author states that the primary purposes of his paper are: (*a*) to give engineers an idea of the difference between the various grades of wood pipes; (*b*) to set forth a standard set of specifications for the assistance of engineers who have had no opportunity to become versed in their design; and (*c*) to safeguard those who contemplate building such pipe. His purposes appear to have been very well accomplished and, on the whole, the paper is well worth careful study by any one who is without information or experience in wood stave pipe design. The writer would issue only one caution to the uninitiated, and that is, that the author appears to give undue prominence to the value of redwood as compared with fir, which may be due to the fact that he may have had less experience with the latter. This tendency will be discussed later. Mr. Moritz.

The writer will confine himself to pointing out some statements which are not exact or which are open to differences of opinion. One of these is that redwood is the best known material for wood pipe. This may have been true 25 years ago, but its accuracy, at the present time, is questioned. There are at least five large companies on the Pacific Coast which manufacture fir pipe exclusively, and only two which manufacture redwood pipe. The writer has no statistics at hand, but it is not unlikely that the output of fir pipe far exceeds that of redwood.

In regard to redwood pipe, the author says:

"Direct exposure to the rays of the desert sun, and alternate wetting and drying when the pipe is used intermittently in irrigation systems, do not lessen its efficiency."

This appears to be an inaccurate statement, but if the author means that such conditions do not hasten the decay of the wood, he will find few engineers to agree with him.

The author cites various specifications of the U. S. Reclamation Service as examples of the embodiment of different fundamentals for the purchase of similar materials, and states that, in some of these, redwood is placed on an equal basis with uncoated fir and, in others, with coated fir. This is an incorrect understanding of the practice of the Reclamation Service, which is to compare redwood, coated fir, and uncoated fir on their merits, as influenced by the peculiar local conditions surrounding each installation. That is, when competitive bids are asked for the three different classes of materials, the prices

* Denver, Colo.

† The discussions by Messrs. Moritz and Miller were presented before the Colorado Association of Members, Am. Soc. C. E., at its meeting of June 9th, 1917.

Mr. Moritz. submitted are not compared directly, but an estimate is made of the probable length of life of each class of construction and the cost of maintenance and replacement, and the ultimate cost is made the basis of decision as to the most economical construction. It is well known that redwood is much more durable than uncoated fir, and it is probable that it is more durable than coated fir under similar conditions, but, as to the latter, available information is not conclusive.

The statement that galvanized bands have been used on wood pipe in the Hawaiian Islands is interesting. The writer did not know that this had ever been done, and he has always held the opinion that the spelter could not withstand the severe treatment given the bands during erection. If galvanized rods can be put in place without serious injury to the spelter, such material would be a great improvement over the asphalt paint generally used, which is not satisfactory. The cost would not be prohibitive.

The author states that machine-banded pipe is made in sizes from 2 to 24 in. Several manufacturers are now making very satisfactory machine-banded pipe up to 30 in. in diameter. The writer is not informed as to whether or not such pipes have been manufactured in larger sizes than 30 in.

The author is not in favor of inserted or slip-joint pipe. Probably few, if any, engineers will agree with him in this. The inserted joint is entirely satisfactory for heads up to about 100 ft., and, if properly reinforced with individual bands, might be used successfully under higher heads. The limit, no doubt, is lower for the larger than for the smaller pipe. Information on this point is not conclusive, but, if the writer may venture a guess, it is that the reinforced inserted joint can be used successfully for 24-in. pipe under a maximum head of 50 ft., and for 8-in. pipe under a maximum head of 150 ft. The reinforced inserted joint has been used successfully on 30-in. pipe under a 25-ft. head. The wood collar is the most vulnerable part of the pipe to the ravages of decaying influences, due to its lack of saturation, and, on this account, it should be used only when necessary.

The author says, "Fir is the pipe that has failed, the oldest lines having been built not more than 10 years, * * *". This is a surprising statement, in view of his knowledge of Mr. Henny's article on "Life of Wood Pipe."* This article does not, by any means, include all the wire-wound fir wood pipes that have been built, but, it does include one pipe that was 20 years old and another that was 19 years old in 1915. The former is stated to be in "good condition" and the latter "in excellent condition, including wire."

Mr. Miller. A. N. Miller,† Assoc. M. Am. Soc. C. E. (by letter).—This paper is a valuable addition to the existing literature on the subject of wood stave pipe.

* *Reclamation Record*, August, 1915.
† Denver, Colo.

The writer cannot agree with Mr. Partridge, however, when he states (page 457) that machine-banded, Class *A* pipe "will have a life of from 15 to 25 years when receiving no attention; a longer life under ideal conditions, as when laid in soils having the least possible corrosive effect on the galvanized wire, and when operating under pressure, so as to insure complete saturation of the wood." This statement may or may not be true. It may be assumed that the life of clear redwood, completely saturated, is exceedingly long, and if the pipe is placed in a soil which permits of the staves being continually saturated, there is no doubt that they will last for 25 years, or even much longer. **Mr. Miller.**

The lack of permanency in machine-banded pipe is due to the corrosion of the wire, which is of comparatively small sectional area. As shown in Table 3, machine-banded pipe is at present manufactured in diameters up to 24 in., with sizes of wire varying from No. 12 in the 2-in. size to No. 4 in the 24-in. size, the spacing of the wires varying with the pressure, as there tabulated.

By referring to Table 2, on continuous wood stave pipe with ordinary band steel, it will be noted that the usual type is made successfully in diameters up to 144 in., and that the corresponding diameter of the band on the 24-in. pipe is $\frac{7}{16}$ in. The wire on the machine-banded pipe is galvanized. On the ordinary pipe, the bars may or may not be; usually, they are not.

It has been the writer's experience that, in the lighter gauges of metal, the corrosion of steel, when exposed to atmospheric and soil conditions, is extremely rapid, even though the metal is galvanized.

Much has been heard lately of the so-called pure irons, which are said to resist corrosion to a marked degree. The advertisements by the manufacturers of these materials would almost lead one to believe that the oxidation of iron and steel is a thing of the past. On close observation, one can readily see that these companies shield themselves behind a good coat of galvanizing. It has been stated recently by a representative of one of the largest manufacturers of this so-called pure iron, which, in reality, is a low-carbon steel made in an open-hearth furnace, that 60% of their tonnage is galvanized. The writer does not wish to give the impression that the product put out by these companies is inferior. He does wish, however, to emphasize the point that, in a practical sense, the virtue, or resistance of this metal to corrosion, is due to the galvanized coating—not to the purity of the base metal, as represented in most of these advertisements. The skill of the press agents employed by these manufacturers is in advance of that of their chemists and artisans. It may be said that in the defeat of corrosion, purity in the base metal is undoubtedly desirable, but the requisite purity is practically or commercially unattainable.

When one studies the subject of corrosion from the point of view of the electrolytic theory, it is readily seen that it is very difficult to

Mr. Miller. manufacture, on a commercial scale, a steel which is non-corrodible. It is practically impossible to eliminate all impurities; it is also difficult to render the steel perfectly homogeneous. As a result of the dissimilarity in the composition of the molecules, electric currents are set up, under certain conditions, which decompose any water present into its constituents, hydrogen and oxygen.

It may be considered that waters carried by wood stave pipes are weak electrolytes, due to the salts held in solution. It must be remembered, also, that ordinary waters are saturated with dissolved oxygen and carry carbonic acid in solution. These waters also have extremely variable degrees of alkalinity. Electrolysis and corrosion, therefore, go hand in hand, ever tending to restore these products to the more stable compounds found in Nature.

The writer has seen extra heavy wrought-iron pipe, in reality steel pipe, which was used to carry Pintsch gas to the railroad cars, rust out completely in 18 months. This pipe was laid in ashes in the railroad yards, and the conditions for rapid corrosion were ideal, the pipe being buried in a relatively concentrated electrolyte. This pipe failed, or rusted through, near the joints where the mechanics had applied their Stillson wrenches in screwing up the pipe, thereby breaking the outer skin. This condition was finally corrected by embedding the pipes in cement mortar. Similar pipes in clay soil have been in use for 15 years without showing serious effects from oxidation.

In the purification of coal and water gas, hydrated oxide, or iron rust, is used to remove the sulphureted hydrogen. The oxide is made from cast-iron filings or machine-shop borings. The filings are piled and wet down with water, and a small quantity of common salt is usually added. In a very few hours great heat is evolved as the chemical action or oxidation proceeds. It is necessary to control the temperature of the mass, as the efficiency of the final product is lowered if high temperatures are permitted. This is done by spreading the oxide in thin layers. The mass is sprinkled from day to day with an ordinary garden hose. It is also turned over periodically so as to expose new surfaces to atmospheric influences. In the course of about 30 days or 6 weeks the oxide is ready for use. This oxide is found to be of slightly different chemical composition from the ordinary precipitated oxide. It is much cheaper, however. The writer cites the foregoing in order that those not familiar with the gas industry may know how quickly the complete oxidation of iron may take place.

It may be stated, therefore, that it is not well to fix arbitrarily the life of a wood stave pipe, at, say, from 15 to 25 years, without knowledge of its location and the chemical characteristics of the soils to which it may be subjected, as the life may be long or short, depending on these governing conditions.

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Paper No. 1412

THE THREE 15-CUBIC YARD DIPPER-DREDGES, GAMBOA, PARAISO, AND CASCADAS, AS SUPPLIED AND USED ON THE PANAMA CANAL*

BY RAY W. BERDEAU, JUN. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. CHARLES EVAN FOWLER, ARTHUR W. MANTON, A. W. ROBINSON, AND WILLIAM M. ROSEWATER.

SYNOPSIS.

The object of this paper is to place before the Society the result of the writer's study of the design, operation, and efficiency of the three large 15-cu. yd. dipper-dredges supplied by the Bucyrus Company for use on the Panama Canal.

The 15-cu. yd. dipper-dredges, *Gamboa* and *Paraiso*, were requisitioned by the Isthmian Canal Commission as part of the permanent equipment of the Panama Canal and for immediate use in completing the channel through Gaillard Cut (formerly Culebra Cut). A contract was made with the Bucyrus Company, which stipulated that the dredges were to be ready for towing to the Isthmus on December 1st, 1913, and January 1st, 1914. The *Gamboa* was accepted at Port Richmond, N. Y., on February 16th, and the *Paraiso*, on April 13th, 1914. The *Gamboa* reached the Isthmus on March 16th,

* Presented at the meeting of September 19th, 1917.

NOTE.—The author of this paper is now in the Military Service of the United States, and is prevented by his duties from contributing a closing discussion.

and was placed in operation on April 4th, 1914, being followed by the *Paraiso,* which arrived on May 22d, and started work on June 7th, 1914. The total cost of the two dredges, and the towing, etc., was $573 287.40, to which initial cost should be added $3 092.50 for the *Gamboa,* and $1 786.21 for the *Paraiso,* the respective amounts necessary to place them in commission after their arrival.

These dredges operated so efficiently that the Isthmian Canal Commission placed another contract with the Bucyrus Company for a third dredge, of improved design, called the *Cascadas,* which was accepted at Port Richmond, N. Y., and successfully towed to the Isthmus, where it arrived on October 21st, 1915, practically 2 months ahead of the promised delivery, and was placed at work in Gaillard Cut on October 31st, 1915, at a total cost of $376 180.

GAMBOA AND PARAISO

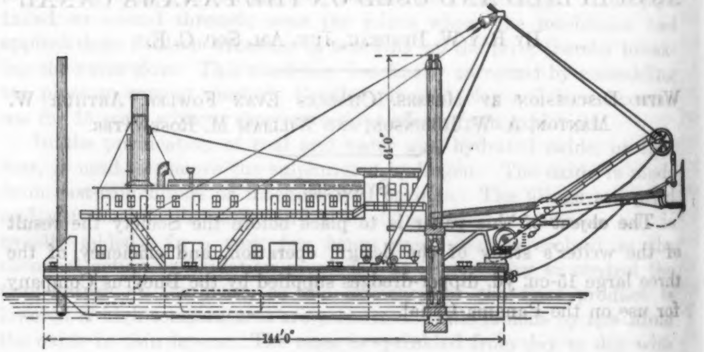


FIG. 1.

The following are the principal dimensions, etc., of the *Gamboa* and *Paraiso*:

Length of hull	144 ft. 0 in.
Beam, moulded	44 " 0 "
Depth, moulded	13 " 6 "
Draft	8 " 0 "
Digging depth, below water line	50 " 0 "
Displacement	1 730 tons.

One main engine, two cylinders, compound, 16 by 28 by 24 in.

One swinging engine, two cylinders, compound, 12 by 16 in.

One backing engine, two cylinders, compound, 12 by 16 in.

Two forward spud engines, two cylinders, compound, 12 by 16 in.

One stern spud engine, two cylinders, 9 by 9 in.

Two deck winches, two cylinders, 6 by 6 in.

Two boilers, Scotch marine type, 126 in. diameter, 138 in. long, water pressure 150 lb.

Two forward spuds, 48 by 48 in., and 82 ft. long.

One stern spud, 30 by 30 in., and 83 ft. 6 in. long.

Swing circle, 24 ft. in diameter.

Bail pull, 235 000 lb.

Hoisting pull on spud rope due to engine, 88 000 lb.

"Pin up" pull on single cable, with brake on engine, 160 000 lb.

Capacity of rock dipper, 10 cu. yd.

Capacity of mud dipper, 15 cu. yd.

Capacity of fuel oil tanks, 14 200 gal.

The displacement of the *Cascadas* is 2 095 tons, and the hull is 144 ft. long, 55 ft. beam, and $15\frac{1}{2}$ ft. deep. Thus, it is 11 ft. wider than the others, making less reactions on the spuds, less metacentric variation when digging over the sides, and it allows the spuds to be inset. The spud-well construction differs from that of the *Gamboa* and *Paraiso,* as their forward spuds are placed outside of the hull, with tapering sponsons fore and aft to transmit the reactions to the sides of the hull.

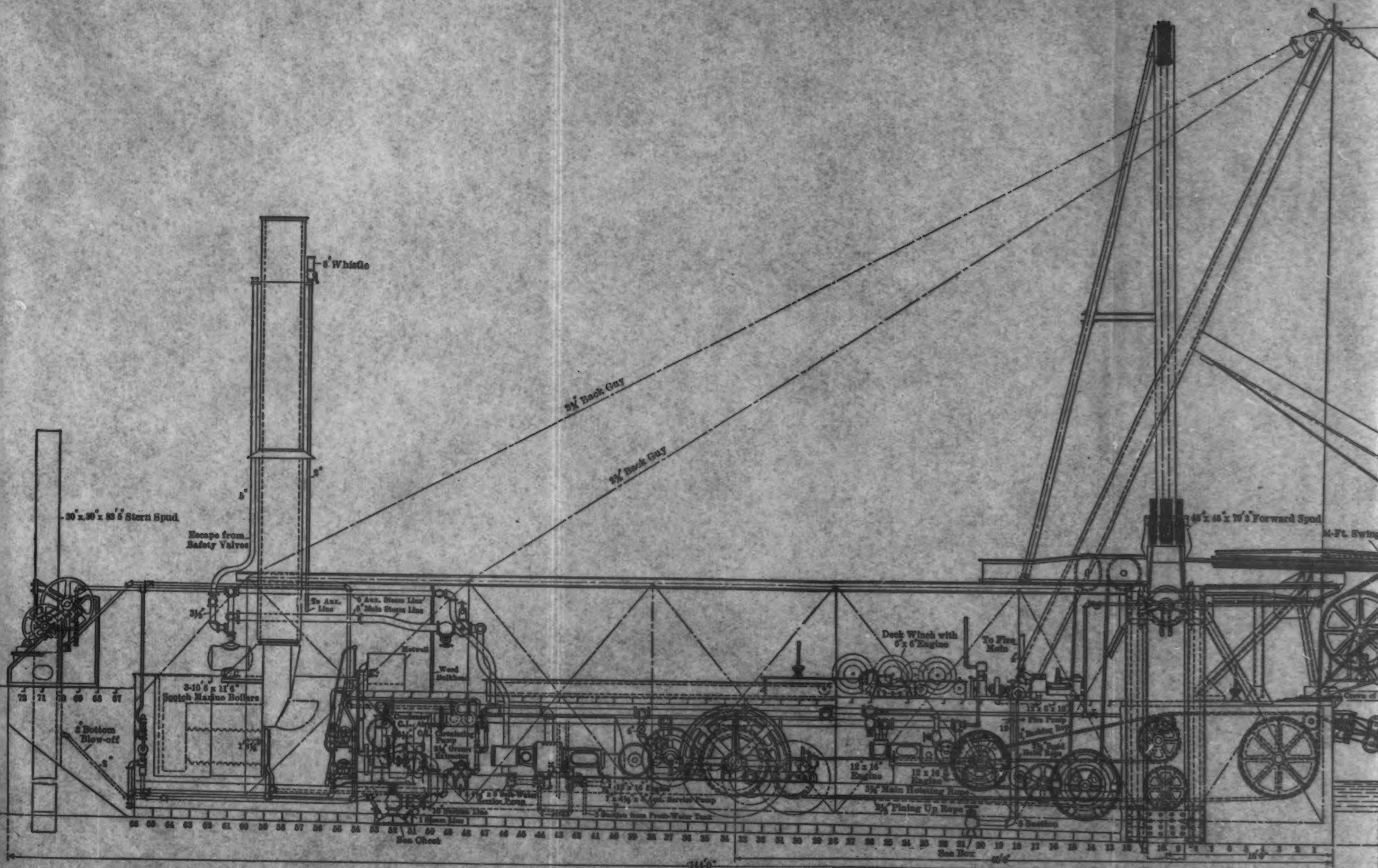
Buckets.—The dredges were supplied with interchangeable buckets of two sizes, one with a capacity of 15 cu. yd. and another of 10 cu. yd., for use in rock excavation. Having been placed in Gaillard Cut in rock digging exclusively, the larger dippers have been seldom used; the smaller ones, as supplied by the contractors, were of extra massive construction, but were of insufficient strength to withstand the severe use and the impact from a dipper stick load of 131 000 lb., and were replaced later by the Missabe type of cast manganese-steel dippers. The over-all dimensions of the new dipper are $10\frac{1}{2}$ by 9 by 9 ft.; the lips are $3\frac{1}{2}$ in. thick at the bottom bands, and the body consists of a front and back casting with lap-riveted joints at the sides; and, in addition, the lip is a separate casting riveted to the front piece and joined thereto by the rivets of the tooth ribs. Recently, the back and bottom of this dipper has been further reinforced, and the dipper

is expected to give greater service and satisfaction than the preceding models.

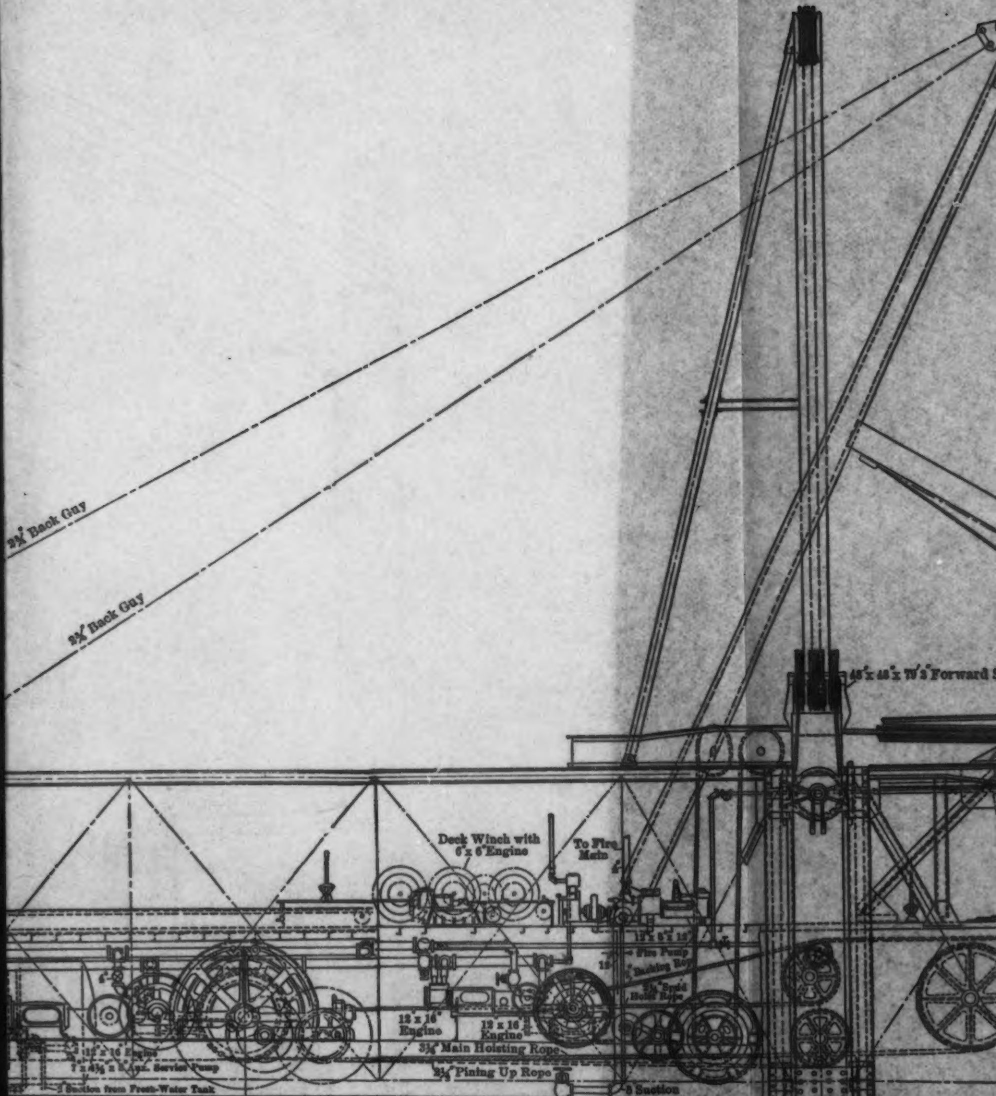
Cables.—The dippers are hung on a 275-ft., 3¼-in., extra pliable, improved, plow-steel wire, the center cable consisting of six strands of 37 wires each, intended to withstand a bail pull of 235 000 lb., the minimum cable life being 3 days and the maximum 35 days. The rapid deterioration of the cables is due to the severe abrasion they receive while at their deeper digging depths (from 35 to 50 ft.), coupled with the deteriorating effects of their constant travel over an undersized point of boom sheave, which is grooved for a wire rope 3¼ in. in diameter and is about 8 ft. in diameter at the bottom of the groove. The sheaves are of cast steel, with long, heavy cast-steel hubs, bronze-bushed, to distribute the pressure over the 11-in. sheave pins, pressed into place; the cables, which are the largest in use on dipper-dredges, are hardly satisfactory for the service required, for which the supplying manufacturers refuse to guarantee them.

Dipper Handles.—The dipper handles on all the dredges are 72 ft. long, over all, and are reinforced top and bottom with 2 by 12-in. bars and by 1 by 22-in. plates on both sides of each dipper stick. Long-leaf yellow pine is used in the construction of the sticks, and white oak for deadwood. The racks are manganese-steel castings, with a pitch of 3 in., and are about 12 in. wide. They are shrouded to the pitch line, the top of the shrouded portion being ground to form a rolling surface for similar shrouding on the rack pinions. Heavy steel castings are used to connect the dipper hinge frame, and similar castings are used to connect the dipper back braces, which are securely bolted to the end of the handle by a large number of 2½-in., horizontal and vertical through bolts. The weight of the dipper handle is 81 000 lb., its life averaging about 6 months, that of the rebuilt handles being 3 months.

Saddle-Block.—The innovation in the design of the saddle-block has proved as useful as it is interesting. The slide plate is separated into two parts for assembling on the flanged shipper-shaft, leaving a passage in the middle, which permits the main hoist cable to run in a straight line from the foot to the point of the boom sheave. This eliminates the usual hump sheave, which is generally placed near the upper end of the boom and is necessary on other dredges to lift the single-hoist cable clear of the saddle-block.



Whistle
Stern Spud
Escape from Safety Valves
Back Guy
Back Guy
Forward Spud
Hotwell
Scotch Marine Boilers
Bottom Blow-off
Deck Winch with 6" x 6" Engine
To Fire Main
Fire Pump
Engine
Engine
Main Hoisting Rope
Piling Up Rope
Sea Chest
Sea Box
144'-0"



2¾" Back Guy
2¾" Back Guy
48" x 48" x 79' 2" Forward S
Deck Winch with 6" x 6" Engine
To Fire Main
12" x 6" x 12"
Fire Pump
Backing Rope
Spud Hoist Rope
12" x 16" Engine
12 x 16 Engine
12" x 16" Engine
Aux. Service Pump
Suction from Fresh-Water Tank
3⅛" Main Hoisting Rope
2⅛" Pining Up Rope
8" Suction
42 41 40 39 38 37 36 35 34 33 32 31 30 29 28 27 26 25 24 23 22 21 20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3
Sea Box
144' 0"

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
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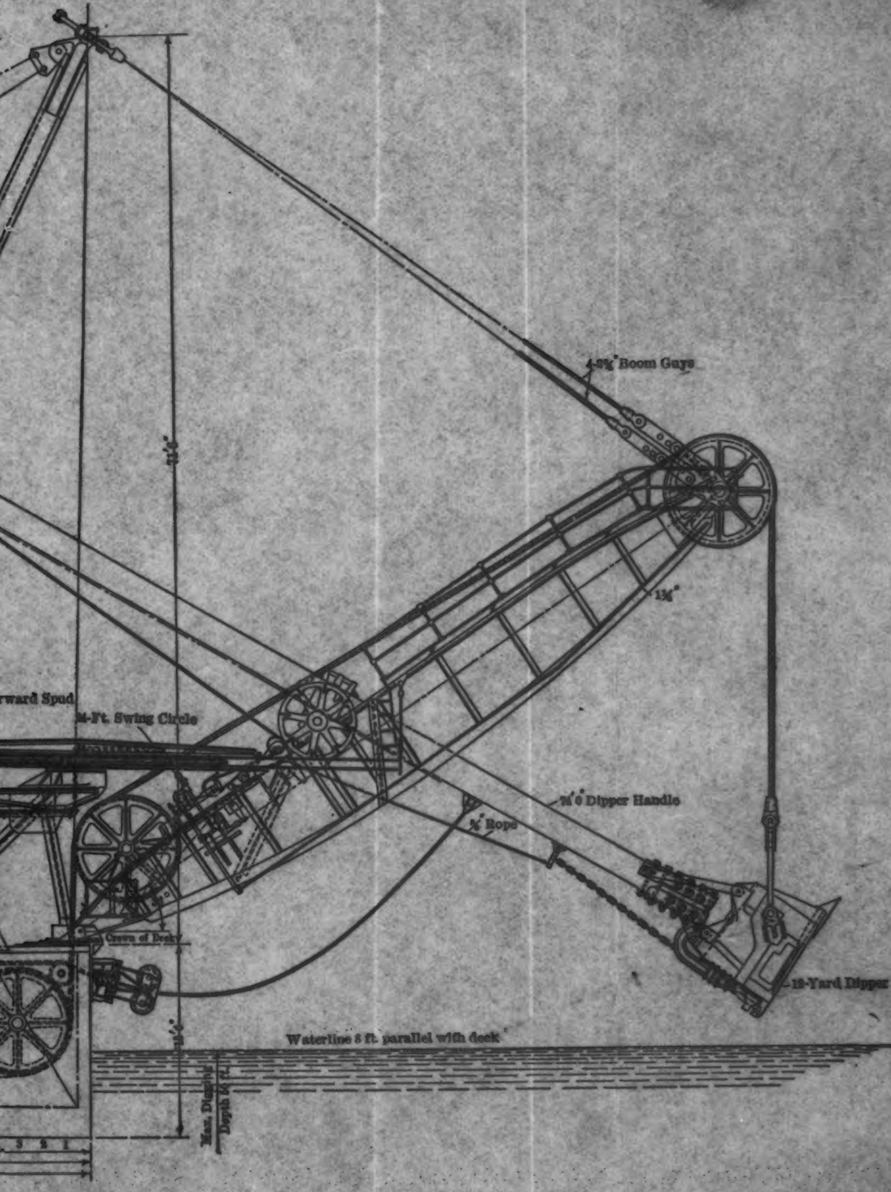
BERDEAU ON
DIPPER-DREDGES
ON THE PANAMA CANAL.



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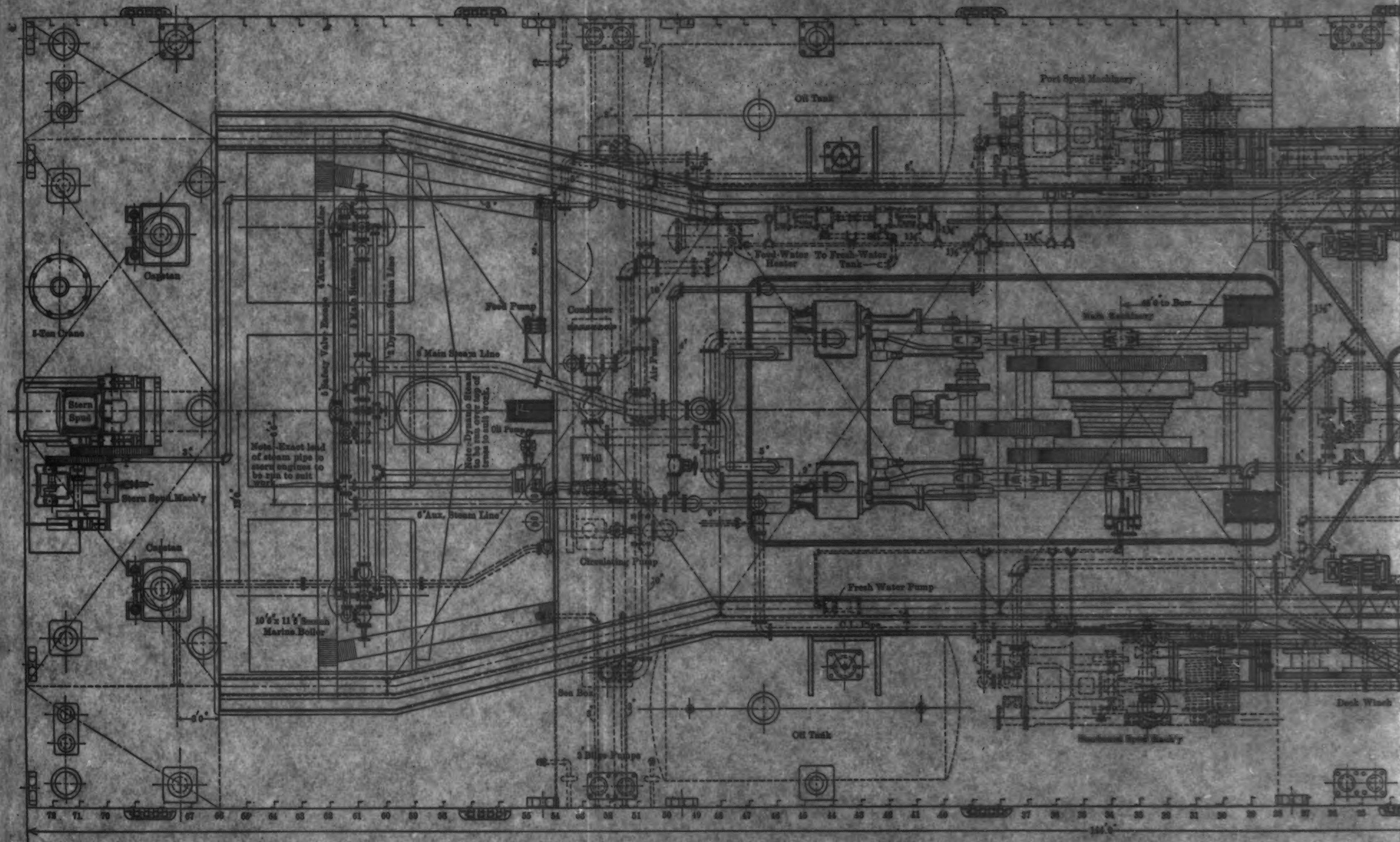
The heavy unit construction facilitates the guiding and holding of the dipper handle in a much more secure manner, and improves the meshing of the racks on the dipper handles with the pinions of the 9½-in. hollow, nickel-chrome-steel, shipper-shaft, at times to such an extent that the teeth are stripped from them both. By building a heavier bucket, the dipper handle, saddle-block, shipper-shaft, and hoisting arrangement offer opportunity for improvement in design, in that the dipper stick would be stronger if it was made of one piece, two main hoisting cables being used, running on each side of the dipper handle, thereby increasing the life of the hoisting cable and also the dipper stick. This dipper stick at times becomes bowed, and, due to the sliding fit with the saddle-block, necessitates immediate re-alignment; this could be obviated if a rolling fit was presented, the rollers being supplied with bearings under compression. The shipper-shaft bearings, which are bolted to the top chord of the boom, project so that the flanges of the brake wheels, which are built of steel castings 75 in. in diameter with a 12-in. face and bolted to the flanges on each end of the shipper-shaft, engage the bearing boxes when lifted for removal, making an extended operation of changing the shipper-shaft. The brakes are of the double-acting type, and are actuated by a steam thrust-cylinder, the steam valve of which is controlled by a floating lever and operated by a hand-lever on the craneman's platform.

Booms.—The booms on these dredges are 62 ft. long, of the plate-girder type, with curved top and bottom chords. All parts of these booms are supposed to be of ample section to withstand developed stresses, which, due to the heavy type of work, have been such as to necessitate reinforcement of the different booms that, with complete machinery, weigh 113 000 lb. They are equipped with a steam-operated boom brake, steam shipper-shaft brakes, and a steam dipper-trip, the cylinder of which is mounted above the foot of the boom, and is connected with the latch bar on the dipper by an endless wire rope, the circuit beginning at the upper end of the dipper handle, leading around the sheave on the stand just below the shipper-shaft, and thence around the sheave attached to the cross-head of the steam dumping cylinder, which permits dumping in any position. The boom feet are of heavy steel castings, with webs and flanges of such length as to permit adequate riveting. The boom is stepped into a steel casting pivot, formed with sockets to receive the boom feet; the pivot

rotates on a heavy cast-steel base plate, securely bolted to the hull, with its flanges extending over the front of the hull. The pintle is bronze-bushed, and has a bronze wearing plate or washer between the boom step collar and the base plate, and another bronze bushing and wearing plate is used between the base plate and the center casting of the swing circle.

Main Engines.—The main engines are specified to work at a steam pressure of 135 lb. while condensing. They are of the horizontal, twin-tandem, compound type, with 16 and 28-in. cylinders and 24-in. stroke, mounted on heavy self-contained cast-iron bed-plates of the Tangye pattern. The crank shaft is of forged steel, 12 in. in diameter, with journals 9¼ in. in diameter and 14 in. long, and with screw-adjusting bearings. The connecting rods, valve stems, and the adjustable bronze-shoed cross-heads are of steel, finished and arranged for taking up wear. The link motion and reverse gears are omitted on the *Cascadas,* and are replaced by a steam turning gear, comprising a steam reversing engine geared to the crank shaft with a releasing jaw clutch operated from the engine-room. The low-pressure cylinders have piston valves working in renewable cast-iron valve cages, a stuffing-box being incorporated between the high and low-pressure cylinders. The eccentric bearings of the *Cascadas* are larger than those of the *Gamboa* and *Paraiso,* and the *Cascadas* is equipped with an overhead 15-ton traveling crane.

Hoisting Drum and Gears.—The hoisting drum is of the differential type; is of cast steel, and is bushed with bronze. The small diameter is 69 in. and the large diameter is 84 in. at the bottom of the grooves, which are for 3¼-in. wire cable. The drum is mounted loose on the 16-in., forged-steel, main hoisting shaft, having journals 11¾ in. in diameter and 18 in. long. Power is applied to the drum by two outside, wood-lined, band frictions, one on each side of the drum, both operated by a single steam cylinder, 14 in. in diameter, placed at one end of the drum shaft and attached to a thrust spindle passing through the center of the shaft. The drum is of the usual type supplied by the Bucyrus Company on its dredges, and has a barrel-like shape, which permits the maximum digging force and the slowest speed when the dipper is excavating and the angle between the hoisting rope and the dipper handle is sharpest; as the rope is wound on the larger diameter, the dipper is hoisted, allowing any desired increase



Oil Tank
Port Spud Machinery
Capstan
5-Ton Crane
Stern Spud
Stern Spud Mach'y
Feed Pump
Condenser
Air Pump
Main Machinery
Oil Pump
Well
Feed-Water Heater
To Fresh-Water Tank
6" Main Steam Line
6" Aux. Steam Line
Circulating Pump
Fresh Water Pump
10'6" x 11'6" Steam Marine Boiler
Sea Box
Bilge Pumps
Oil Tank
Deck Winch

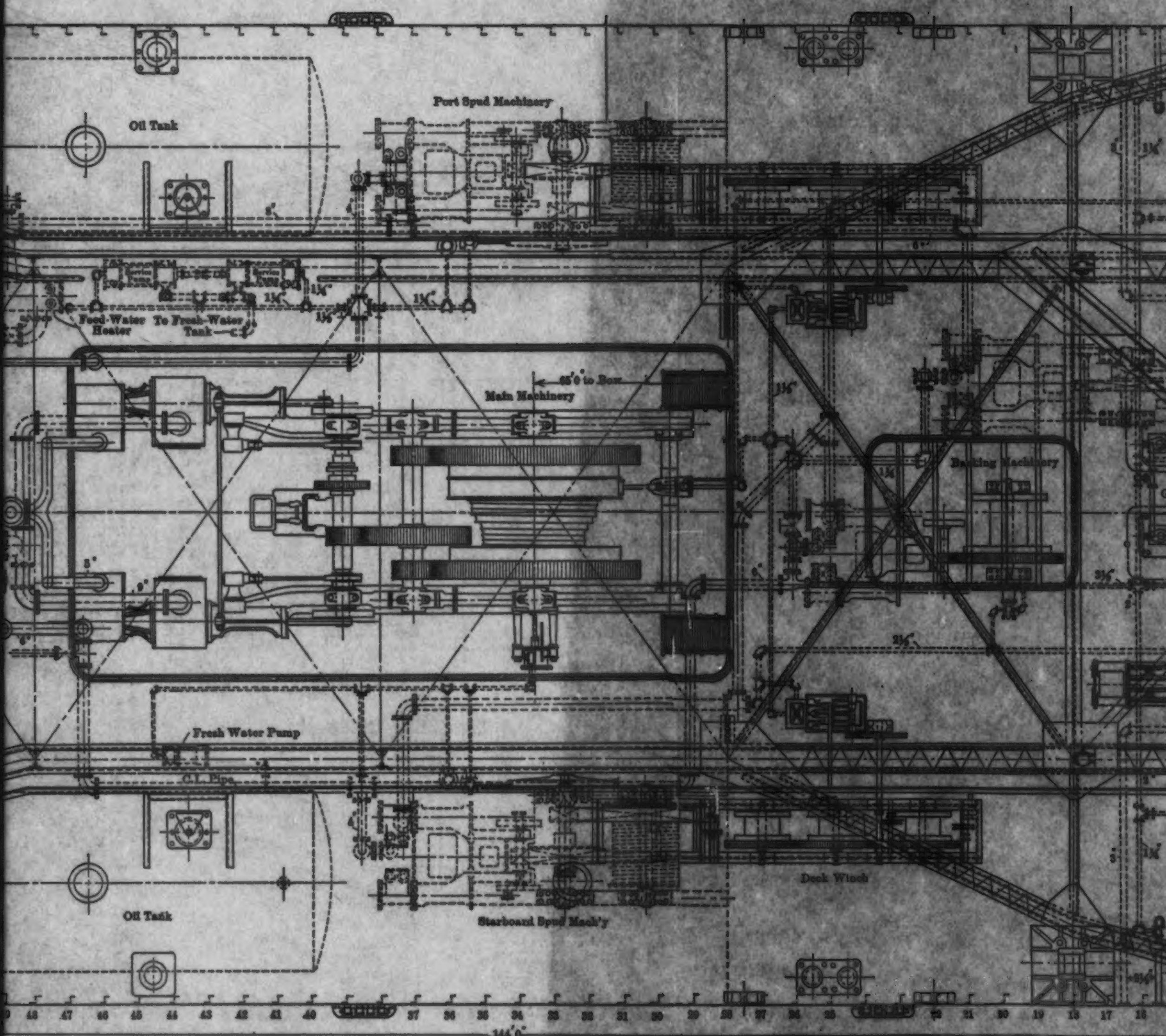
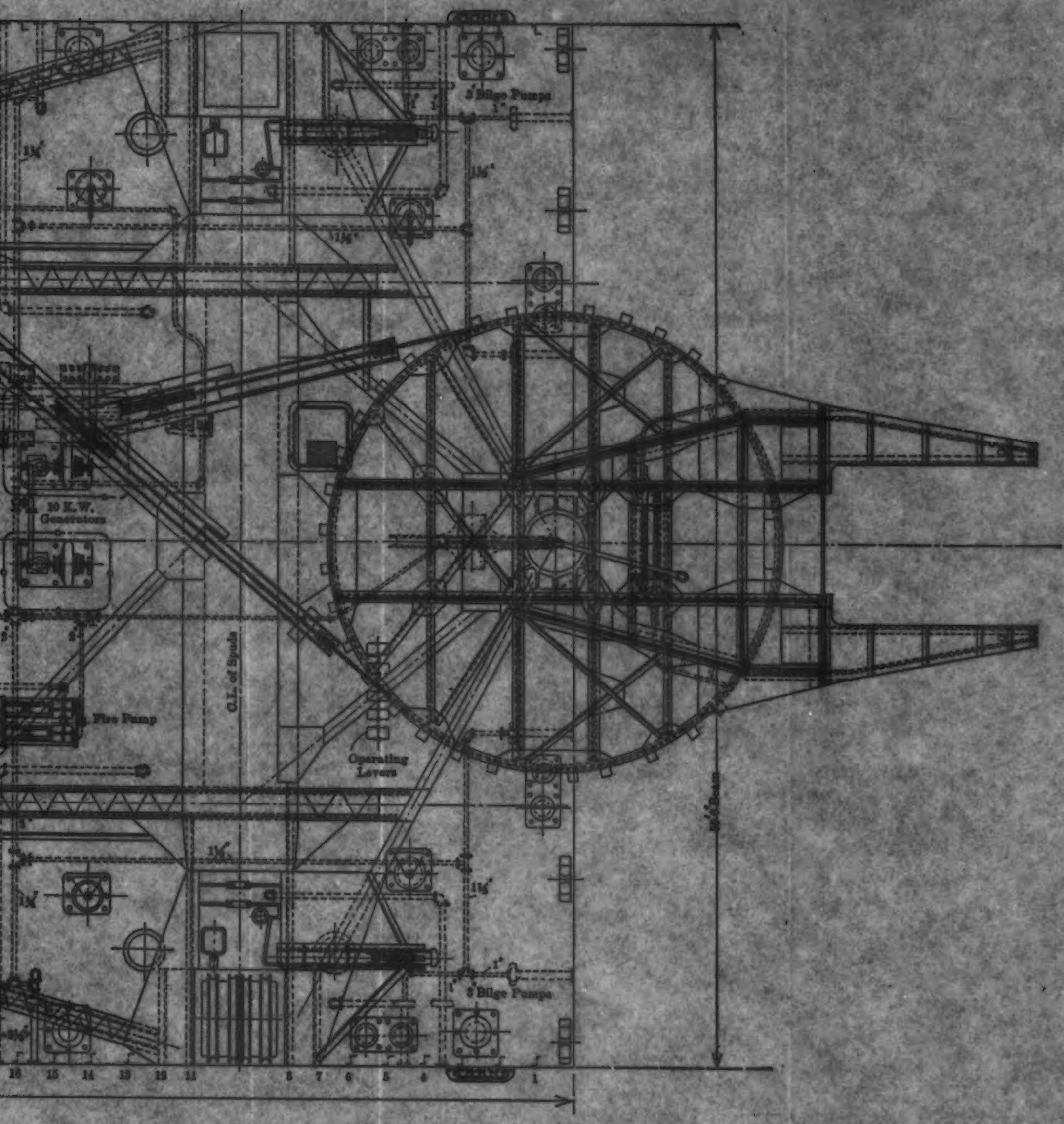

Oil Tank
Port Spud Machinery
Feed-Water Heater
To Fresh-Water Tank
65'0" to Bow
Main Machinery
Backing Machinery
Fresh Water Pump
C.I. Pipe
Oil Tank
Starboard Spud Mach'y
Deck Winch
144'0"

PLATE VII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1412.
BERDEAU ON
DIPPER-DREDGES
ON THE PANAMA CANAL.



of hoisting speed when the maximum bail pull is not required. The diameter of the drum could be increased, which would reflect in length of cable life. The drum is driven by two heavy spur-gears, 12 ft. in diameter, one on each side, meshing with the corresponding pinion of the intermediate shaft. The gear hubs next to the bearings are lined with bronze. The intermediate shaft is driven from the crank shaft through a single gear, which has steel castings rim-bolted to a heavy cast-steel spider arranged for rim replacements without stripping the shaft. The intermediate gear rim is split for easy removal and replacements, and, like the pinion on the crank shaft, has cut teeth. The intermediate shaft is bolted directly on the engine bed-plates; the bed-plate also contains the drum-shaft bearings, and is of cast steel, forming an extension of the engine bed-plate, and is securely bolted to it and the structural base built into the hull.

Swinging Circle Machine and Guide-Sheaves.—The swinging circle is of structural steel, 24 ft. in diameter, and mounted on top of the hull truss; connection is made to the boom with two heavy built-up girders, extending out from the circle, one on each side of the boom. The center is a heavy steel casting, securely bolted to the circle, and has an I-beam rim reinforced with ½-in. plates, the jaws being fastened to the boom at the forward ends. Changes have been effected in the rope anchorage of the dredges by making the swing rope around the circle in four separate pieces with open socket connections, which renders complete stripping unnecessary when the rope is changed. Two 42-in. cast-steel sheaves, on top of the hull truss, grooved for a 2-in. rope, and complete with shaft bearings and 4⅜-in. sheave pins, are supplied for guiding the swing rope from the circle to the swinging drum. The swinging machinery is operated by an independent double engine, with 12-in. cylinders and 16-in. stroke, and reversing link gear. The engine and drum are mounted on heavy structural steel bases built into the hull, and the links are reversed by the steam thrust-cylinder controlled by the lever which operates the throttle.

Backing Engine.—The backing engine and drum are mounted on a structural steel base built into the hull, and the drum is operated by a separate 12 by 16-in., double, non-reversing engine. The cast-steel drum (26 in. pitch diameter) is driven by an outside-band friction clutch, actuated by a steam thrust-cylinder, 5 in. in diameter, and carries a 2-in. steel rope. The diameter of this drum is too small,

as it breaks the strands of the cable, prevents proper reeling, and causes slack on the drum, which prohibits uniform backing of the bucket. The gear reduction between the crank and the 7-in. drum shaft is single, and a band brake prevents the running out of the rope.

Forward Spud Machinery.—Each forward spud is operated by an independent, double, 12 by 16-in. engine, with link motion reverse, which is operated by a steam thrust-cylinder controlled by the lever which operates the throttle valve. The spud drums are 42 in. in diameter and are grooved for 2¼-in. wire rope. The *Cascadas* is "pinned up" and the spuds are lifted by a 2¼-in. wire rope passing around the four sheaves at the top of each spud; the pinning-up ropes are similar to those on the *Gamboa* and *Paraiso,* but the spud hoist ropes run over sheaves which are on a gantry mounted near the spud casings and extending above the highest position of the spuds.

The Bucyrus Company has developed a big improvement in this design, as it dispenses with the sheaves at the lower end of the spuds, where the wire ropes are quickly cut by the sharp stones found in the rock excavation in "the Cut." Wood-lined band-brakes, operated by a steam cylinder, are supplied to hold the dredge when "pinned-up", and the spud tackle rope is taken up by four parts of rope. Each spud engine is connected to its drum by double reduction gearing of cast steel; and a suitable friction clutch, operated by a steam thrust-cylinder, is provided to disconnect the drum from the engine, allowing the dredge to rise and fall with the tide when not in use. The spud drum machinery and engines are supported by a heavy structural steel frame built into, and set far enough from the side of, the hull to permit ready access to the spud machinery on the outboard side of the foundation. The sheaves guiding the spud ropes to the drum have a pitch diameter of 45 in., are cast steel, bushed with bronze, and have annealed, forged-steel, sheave pins. The stern spud machinery is arranged for a trailing spud that is hoisted by rack and pinion, through two gear reductions, by a 9 by 9-in., double engine, placed on and operated from the deck, at the stern of the dredge. The intermediate shaft is fitted with a band-brake, to hold the spud in its proper position, and carries a jaw clutch for disconnecting the drum from the shaft and the engine. The stern spud is of structural steel, 30 by 30 in. and 83 ft. 5 in. long, and is held in place by a sliding-fit arrangement somewhat similar to that

of the saddle-block. This creates the necessity of immediate re-alignment when the spud becomes slightly bowed. The forward spuds are of structural steel, 48 by 48 in. and 72 ft. long, with 8 by 8 by ¾-in. corner angles, ¾-in. side plates, and suitable diaphragms, those of the *Cascadas,* as mentioned, having all sheaves at the top of the spud. As heavy as they are, these spuds have to be removed every 90 days, three being broken in as many days, in one case.

Deck Fittings.—On the main deck, outside of the house, there are two three-drum winches, which are used for moving scows, and are equipped with internal driving engines. Each engine is of the horizontal type, and has two 6-in. cylinders with 6-in. stroke; the throttle-valve is in the main steam chest, and also acts as a reversing valve. The drums are steel castings, bushed with bronze, with 24-in. barrels 12 in. long, and equipped with friction clutches of the outside-band type, as well as band-brakes. There are ten double mooring bits, ten single deck spools, and sixteen deck chocks.

Boilers and Fittings.—Steam is provided in the *Cascadas* by a battery of three boilers of the Scotch marine type, arranged so that any two boilers can be used at one time, the third one being a spare, permitting of blowing down without tying up the dredge, this being an additional boiler to the two supplied, respectively, to the *Gamboa* and *Paraiso.* The boilers are constructed for a working pressure of 150 lb. per sq. in.; each is supplied with two Morrison suspension furnaces, and is equipped with marine pop safety-valves, stop valves, pressure gauges, blow-off cocks, etc. The stack is double, and about 50 ft. high above the tops of the boilers. One surface condenser is supplied, having approximately 1 500 sq. ft. of cooling surface, together with an independent air pump and a 10-in. brass-runner, centrifugal, circulating pump, driven by a separate engine. One 300-h.p., feed-water heater of the closed type and two 7 by 4½ by 8-in. brass-fitted, horizontal, duplex, boiler feed-pumps are provided, also a similar pair for general service, one fresh-water, 4½ by 2¾-in., horizontal pump, one 12 by 6 by 12-in., horizontal, duplex, brass-fitted pump, and a bilge pump for each compartment. The piping system is very complete, each line having a stop-valve to cut it off from the boiler or main, so that repairs can be made at any time without shutting down the system.

All the large pipes are flanged, and the steam fittings for 200 lb. pressure and exhaust fittings for 125 lb. are extra heavy.

Tanks.—There are four steel fresh-water tanks in the stern of the hull; two of 30 000 gal. capacity are supplied for boiler-feed purposes, on the port and starboard sides, respectively. There is an 8 000-gal. tank on the port side for galley use and a duplicate tank on the starboard side for ballast and trimming. There are two 18 000-gal. oil tanks in the hull forward of the boiler, one on each side of the dredge, and two oil-feed pumps are used to supply oil to the boilers.

Electric Light Plant.—Two 10-k.w., 110-volt, direct-current generators, each connected directly to a vertical, automatic engine, are provided for lighting the dredge, operating the house lights, the searchlight, and the lights used in night dredging.

House and Cabin Construction.—The main deck house is 108 ft. long and 41 ft. wide from the front end to Frame 48 (about 95 ft.), and 30.4 ft. from Frame 48 to the after end. The height is about 17 ft. for a distance of 18 ft. from the bow end, and 15 ft. throughout the remaining length. The upper deck over the house is 112 ft. long and 44 ft. wide throughout, projecting 6 ft. 9 in. over the lower deck house on all sides. Quarters are supplied for 11 "gold" (white) American employees and 57 "silver" (colored) employees. Three officers' staterooms have hot and cold-water service, one berth and single wall lockers. All other staterooms are fitted with double, wooden, built-in berths, with two drawers and lockers under each lower berth, double, wooden wall lockers, and wash-basins with hot and cold water. The galley and dining-room are amidships, between the "gold" and "silver" quarters; in the latter, thirty-eight standee bunks with canvas bottoms, and thirty-eight sanitary steel lockers are supplied. There is also separate modern bath and toilet service on the main deck for both "gold" and "silver" men.

The Hull.—The hull of each dredge is of steel. The *Cascadas* is 144 ft. long over all, 55 ft. wide, and has a mean depth of 14 ft. 6 in. (15 ft. 6 in. at bow at center, and 13 ft. 6 in. at stern at center), with a straight taper fore and aft. The deck has a 6-in. camber made by straight lines from the center to each side.

Operation.—All three dredges are 15-cu. yd. machines, built by the Bucyrus Company, in the United States, and have been working until recently in Gaillard Cut of the Panama Canal. The material

TABLE 1.—MONTHLY PERFORMANCE OF DREDGE *Gamboa*.

Date.	Cubic yards.	Cost of operation.	Cost of maintenance.	Total cost.	Cost per yard.
1914.					
Apr	32 805	$4 446.82	$10 798.18	$15 245.00	$0.4647
May	108 185	7 365.53	16 165.11	23 530.64	0.2175
June*	123 199	6 764.68	24 279.25	31 043.93	0.2519
July	108 896	7 441.27	21 148.06	28 589.33	0.2625
Aug	121 850	7 002.58	14 772.77	21 775.35	0.1787
Sept	111 355	6 561.27	12 238.25	18 799.52	0.1688
Oct	137 060	7 377.96	9 015.69	16 393.65	0.1205
Nov	140 905	7 612.19	8 902.44	16 553.81	0.1174
Dec	166 092	8 147.03	12 997.82	21 144.85	0.1274
1915.					
Jan	178 370	7 823.27	14 351.67	22 147.94	0.1244
Feb	149 554	6 308.69	8 939.04	15 247.73	0.1020
Mar	207 870	7 425.32	9 643.36	17 068.68	0.0821
Apr	168 725	7 003.25	10 593.31	17 596.56	0.1043
May	165 720	7 599.38	11 839.11	19 438.49	0.1173
June*	168 725	7 075.09	11 332.41	18 407.50	0.1091
July	171 370	7 376.09	9 421.10	16 797.19	0.0980
Aug	199 425	8 102.46	12 111.33	20 213.79	0.1014
Sept	280 550	8 263.33	8 455.81	16 719.14	0.0596
Oct	249 515	7 610.10	14 612.47	22 222.57	0.0891
Nov	260 845	8 280.17	10 431.38	18 711.55	0.0717
Dec	266 470	8 877.27	9 624.75	18 502.03	0.0694
1916.					
Jan	232 855	7 515.06	8 590.39	16 105.45	0.0692
Feb	293 230	9 103.00	10 056.95	19 159.95	0.0653
Mar	304 006	9,241.50	10 790.25	20 031.75	0.0695
Apr	263 275	8 896.18	10 460.12	19 356.30	0.0735
May	271 235	8 638.56	8 996.67	17 635.23	0.0650
June*	304 450	9 127.41	12 003.65	21 131.06	0.0694
July	320 190	9 448.95	9 055.02	18 503.97	0.0578
Aug	129 210	6 221.29	8 046.99	14 267.78	0.1108
Sept	150 175	7 748.87	8 238.84	15 987.71	0.1063

YARDAGE EXCAVATED BY FISCAL YEARS.

July 1st, 1913, to July 1st, 1914.					
	264 189	....	....	$69 819.57	$0.3021
July 1st, 1914, to July 1st, 1915.					
	1 825 122	....	....	233 190.41	0.1278
July 1st, 1915, to July 1st, 1916.					
	3 097 226	....	....	226 586.01	0.0731
July 1st, 1916, to October 1st, 1916.					
	599 575	....	....	48 759.46	0.0713

* End of fiscal year.

TABLE 2.—MONTHLY PERFORMANCE OF DREDGE *Paraiso*.

Date.	Cubic yards.	Cost of operation.	Cost of maintenance.	Total cost.	Cost per yard.
1914.					
June*	69 812	$9 002.69	$11 458.21	$20 460.90	$0.2931
July	82 203	5 717.11	10 221.48	15 938.59	0.1939
Aug	95 475	6 728.67	20 155.38	26 884.05	0.2816
Sept	105 470	7 236.62	17 889.63	25 126.25	0.2382
Oct	125 605	7 380.00	9 644.65	17 024.65	0.1355
Nov	139 916	7 938.98	15 110.26	23 091.19	0.1650
Dec	144 165	7 824.00	12 371.43	20 195.43	0.1401
1915.					
Jan	176 862	8 111.45	11 380.86	19 492.31	0.1102
Feb	172 057	5 617.64	9 310.49	13 928.13	0.0868
Mar	134 030	6 395.02	8 740.37	15 135.39	0.1129
Apr	204 250	7 410.48	9 274.15	16 684.63	0.0817
May	184 510	7 239.92	10 889.73	18 129.65	0.0983
June*	174 685	7 880.58	8 855.95	16 766.53	0.0960
July	170 060	8 003.89	9 970.81	17 974.70	0.1057
Aug	203 815	8 064.34	10 748.12	18 812.46	0.0923
Sept	220 940	7 280.87	10 986.89	18 267.76	0.0827
Oct	291 675	8 859.52	15 502.24	22 361.76	0.0767
Nov	262 925	8 740.54	9 193.33	17 933.87	0.0682
Dec	312 920	9 716.54	10 153.78	19 870.32	0.0635
1916.					
Jan	195 515	7 295.94	8 587.49	15 883.43	0.0812
Feb	236 235	8 710.18	11 975.35	20 685.53	0.0876
Mar	299 155	9 099.70	11 323.76	20 423.46	0.0683
Apr	232 365	8 887.67	10 152.90	19 040.57	0.0819
May	306 435	8 649.36	10 871.43	19 520.79	0.0637
June*	272 064	9 115.15	11 275.58	20 390.73	0.0749
July	233 930	6 418.34	7 192.01	13 610.35	0.0581
Aug	315 915	9 823.62	12 505.83	22 329.45	0.0714
Sept	268 250	8 921.49	9 029.42	17 950.91	0.0669

YARDAGE EXCAVATED BY FISCAL YEARS.

July 1st, 1913, to July 1st, 1914:					
	69 812	....	....	$20 460.90	$0.2931
July 1st, 1914, to July 1st, 1915:					
	1 739 228	....	....	228 396.80	0.1313
July 1st, 1915, to July 1st, 1916:					
	3 004 104	....	....	231 165.38	0.0769
July 1st, 1916, to October 1st, 1916:					
	818 095	....	....	53 890.71	0.0658

* End of fiscal year.

TABLE 3.—MONTHLY PERFORMANCE OF DREDGE *Cascadas*.

Date.	Cubic yards.	Cost of operation.	Cost of maintenance.	Total cost.	Cost per yard.
1915.					
Oct.	695	$427.10	....	$427.10	$0.6145
Nov	296 280	9 371.94	$10 056.25	19 428.19	0.0656
Dec	321 065	9 961.47	10 221.35	20 182.82	0.0628
1916.					
Jan	292 675	9 054.96	10 769.15	19 824.11	0.0677
Feb.	330 605	8 982.54	9 939.51	18 922.05	0.0573
Mar	309 125	8 796.39	10 723.06	19 519.45	0.0632
Apr	246 786	8 649.02	10 775.53	19 424.55	0.0787
May	316 770	8 848.07	10 818.53	19 666.60	0.0621
June*	286 491	9 171.57	9 995.13	19 166.70	0.0669
July	340 185	9 383.79	10 244.25	19 628.04	0.0577
Aug	208 967	8 135.09	9 481.83	17 616.92	0.0862
Sept	122 504	6 864.80	5 367.74	12 232.54	0.0987

YARDAGE EXCAVATED BY FISCAL YEARS.

July 1st, 1915, to July 1st, 1916.

	2 400 492	....	....	$156 561.57	$0.0651

July 1st, 1916, to October 1st, 1916.

	666 656	....	.. ..	49 477 50	0.0742

* End of fiscal year.

excavated consisted of hard and soft rock, to a depth of from 35 to 47 ft. When the scows are moored on the port side of the dredge, their loading is sometimes handicapped by the inadequate view obtained by the operator while he works, as direct vision of the mid-scow pockets is obstructed by the structural frame supporting the boom and the port forward spud; these pockets, when empty, necessitate the lowering of the bucket into the pocket, so that the spoil when dumped, will not damage the bottom doors. An arrangement of an ordinary mirror, about 24 by 14 in., on the operator's right helps to a certain extent, but, as the scow in the view does not appear quite natural, the scow strong backs suffer. Reflecting mirrors could be arranged to give a direct full view of the loading section, thereby enabling quicker operation. The *Gamboa* and *Paraiso* assisted in raising the drill barge, *Teredo*—which was blown up and sunk in the channel at the foot of Cucaracha Slide on July 20th, 1914—by attaching the main hoist cables to slings and raising the wreck. Tables 1, 2, and 3 give the

yardage of the respective dredges, *Gamboa, Paraiso,* and *Cascadas,* and the material placed in scows alongside the dredges. The accompanying costs include operation, that is, wages of crew, subsistence of crew, fuel, and lubricants, maintenance, that is, the cost of keeping the equipment in first-class physical condition, and depreciation only. Extra heavy 10-yd. manganese-steel dippers were used on this work, the dredges working continuously in three 8-hour shifts, under the charge of the Resident Engineer, W. G. Comber, M. Am. Soc. C. E., and the Superintendent of Dredging, Mr. James Macfarlane.

DISCUSSION

Mr. Fowler.

CHARLES EVAN FOWLER,* M. AM. SOC. C. E.—There has been no other kind of engineering design in which the practical problems have had larger scope than in that of dredges of all kinds, and particularly is this true of the design of dipper-dredges. For many years the speaker, as chief and consulting engineer, has been connected with the dredging of a vast quantity of material in the Pacific Northwest. This material has consisted of a greater percentage than usual of hardpan and cemented gravel, which is harder to dig than many kinds of soft rock, and offers greater resistance, for all parts of dredging machinery, than any material of which the speaker is aware.

Some years ago, in building a dredge for digging this material, the machinery complete was purchased for a 7-yd. dipper plant, and, as the time in which to assemble the dredge was short, it was mounted on an old clam-shell dredge hull, 50 by 100 ft., and the compound engines on this were used as swinging engines by connecting them to the turn-table with double wire rope blocks. Many other makeshifts had to be used, but, after the rush work was completed, a new wooden hull was built, which was 40 by 100 by 11 ft., and proved to be none too large for dredging of this class. There is nothing more important than having a hull with sufficient beam for side digging, to take the load off the spuds, and, of course, the hull must be of such length that it will not rise out of the water at the stern when digging hard material or when digging with a large dipper. It will readily be seen from this that the first two Panama dredges, which had a beam of only 44 ft., were too narrow, and it was certainly a wise change to make the *Cascadas* of 55 ft. beam.

Even when ordinary conditions as to getting steel prevail, the speaker has found many advantages in building wooden hulls, inasmuch as it is often necessary to cut through the hull for various purposes, or to make various connections to it, all of which can be more readily done with a wooden hull. Then, again, a wooden hull is more easily repaired than a steel hull, and the speaker knows of wooden hulls which are 30 years old, are still in use, and are giving good service. However, such hulls must be properly built, so as to have sufficient strength and stiffness and plenty of air space; and, in salt water, they must be properly constructed for protection against the teredo. Under present conditions, the speaker thinks that, for all classes of dredges, hulls constructed of reinforced concrete would be cheaper than either wood or steel, and in many cases preferable. Many recent dredges of all types have been constructed with steel hulls, and some have been used on work with which the speaker has been connected, these dredges

* New York City.

Mr. Fowler. also having steel spuds and everything of the most substantial construction.

No matter how exactly everything has been figured out for dipper-dredges, especially as regards the boom and dipper handles, it will be found that it is necessary, almost immediately, to begin re-driving rivets and reinforcing the sections; therefore, it would seem to be only a matter of precaution to assume that the loads which cannot be figured should be taken care of by at least doubling the calculable stresses for loads suddenly applied. The dippers are the most frequent sources of trouble, and have to be reinforced, particularly on the front and around the nose. The same thing happens with almost all the clam-shell buckets of standard design, as it is usually necessary, not only to increase the weight of the shells, but of all the pins and bearings as well. With reference to sheaves, it has been the speaker's practice to make them as large as reasonably possible, in order to lengthen the life of the wire ropes, and to provide them with very large pins and boxes in order to avoid cutting the bearings.

The discussion on this paper has brought out the fact that there is grave doubt as to whether compound engines are desirable for dipper machines, and there is certainly no point about dredge design about which the purchaser should be more certain as to capacity than in the boilers and engines. The speaker has found it very satisfactory to check up the engines by Seaton's formula, which gives very close to the actual indicated horse-power, although it very often calls for an engine of much larger size for the required horse-power than would be offered by an engine builder.

For nearly twenty years the speaker has operated fleets of dredges, comprising all the different types, and it is largely a matter of judgment as to the type which should be used in any particular work. The plant of the United Dredging Company, for which the speaker is at present acting as Consulting Engineer, comprises suction hopper sea-going dredges; regular suction-dredges of the largest type; suction-dredges which are provided with propellers for going from one job to another under their own steam; dipper-dredges; and large clam-shell dredges. This gives the necessary variety to handle work of all kinds in the United States, Alaska, Mexico, Hawaii, and abroad.

The discussion has also brought out the fact that the cost given in the paper does not cover the cost of the disposal of the material, and this should be very strongly emphasized in the paper before its final publication, as it is very important to have dump-scows of proper number and size, as well as tugs of sufficient power, in order to get a project started, and take out the full yardage of which the dredge is capable. The speaker is always skeptical in regard to published costs of dredging work, as the figures are usually only for the bare

dredging, without the proper overhead charges and extensive plant repair, which may cost as much as from $20 000 to $50 000 at a time, if a job has lasted from 6 to 12 months, and which certainly bears no relation to the ordinary figures which are included for simple depreciation. Mr. Fowler.

Some years ago, when making prices on dredging work in the Northwest, the speaker was bothered with reports of dredging being done on the Columbia River at from 2 to 3 cents per cu. yd. This was investigated fully, the total costs for a number of years previous —back to the inception of the work—being taken from the reports of the Port of Portland, including overhead, experimental work, and expenditures of every sort. The yardage reported was also taken, although a large part of this was admittedly washed out by the current; and the cost was found to be more than 26 cents per cu. yd. This, of course, has been greatly bettered during recent years by more efficient administration and a more efficient and modern plant, but, from this source no more is heard of ridiculously low costs. Some one must pay general or overhead expenses eventually.

In closing, the speaker would like to call attention to a type of strongly built suction-dredge, provided with a good type of rock cutter, which can do very efficient work in material often excavated with a dipper-dredge or some other type. Modern high-powered suction-dredges with booster pumps in the pipe line have been used to pump to distances of 20 000 ft., or nearly 4 miles, and the modern design of dredging pumps, with from 1 200 to 1 500 h. p., are capable of pumping through very long pipe lines, ranging from 4 000 to 7 000 ft., without boosters; and to pump to very much greater heights than was believed possible a few years ago. In one recent case, the speaker delivered material with a pump of this type to a distance of about 4 500 ft. on a lift of 45 ft., the material being heavy sand with some clay. The places where dipper-dredges of very large size, or dredges of any type of large size, can be used economically are comparatively few, and the engineer must make sure of all his conditions before deciding on the type, or he must leave it to a contractor of great experience to select the kind of plant and let him take his own chances.

Arthur W. Manton,* M. Am. Soc. C. E. (by letter).—There are a few points relative to these dredges about which the writer would like to have more information, although, perhaps, it may not be considered within the scope of the paper. Mr. Manton.

The writer spent some time on the dredges at the Pacific end of the Panama Canal, with James Macfarlane, Superintendent of Dredging, and was much interested in the performance of the Lobnitz bucket-and-ladder dredges built in Scotland for the French Company. The work

* New York City.

Mr. Manton. done by these old dredges, although in softer material, was considered to be excellent, and, as regards cost, very satisfactory. Since then, the writer has also had the opportunity of studying the work performed by the bucket-and-ladder dredges on the Suez Canal.

After noting the very considerable and expensive repairs and renewals (materials and labor) on the three dredges, as stated in the paper, the writer cannot help thinking that perhaps the advantages—from some "angles"—of the bucket-and-ladder type of dredge may have been overlooked in adopting the dipper-dredge design; and it is hoped that the author will give the reasons for the adoption of the latter. Apparently, the stresses on the boom, dipper handle, spuds, and hull of the three dredges were enormous; and, in order to deal with large masses of rock, it appears that something had to be sacrificed.

The bucket-and-ladder dredges which the writer has operated abroad had large outputs, with small repairs and maintenance charges, both as to labor and material, throughout extended periods of work, although they were subject to severe stresses in a seaway, due to bad weather; in one case, especially, at Portsmouth Dock Yard, one of the dredges was moored and working in the seaway, and had to be hauled out of its "cut" very rapidly in order to permit battleships to pass in and out of the yard. No spuds were provided, and, in this respect, American practice seems to be excellent. If the dredge had been provided with a rear swinging spud, the work would have been improved. The channel bottom had to be dredged to a depth which required an accuracy limit of about 6 in. The material was compact gravel, although not specially hard to dig, and the face varied from 3 to about 12 ft. The buckets had a capacity of about 27 cu. ft., and were operated at about 11 per min., the dredge capacity—a barge-loading dredge—being between 500 and 600 cu. yd., *in situ,* per hour.

Under the circumstances, the cost figures were distinctly reasonable, although they would have been considerably less had the dredge been able to operate for the full 24 hours, instead of at low tide—daylight hours only—as the Admiralty would not permit night dredging in the fairway.

Referring again to the cost of the three "dippers", the writer would be glad if the repairs-and-maintenance cost (divided into materials and labor) could be separated from the actual operation cost, and if the costs of fuel and wire rope could be given separately; it is inferred that the latter was very high. He would also like to know what was provided for depreciation. It is assumed that the staff superintendence cost, insurance cost, and general charges are not included in the figures given; the operation time-losses per 24 hours for each cause would also be interesting.

The maintenance cost (nearly 150% of the operation cost), appears to be high, and this would lead one to suppose (although this point was

not brought out so fully by discussion perhaps as it might have been) that the dredge was operated in a way not at all anticipated; or that the rapid completion of the Gaillard Cut was considered to be so important that it was a small matter to run the dredge in such a way as to distress it in a manner not provided for by the manufacturers, or, indeed, the engineers. This point was only brought out by Mr. Rosewater, but from the paper the dredges may be said to have worked exceedingly well under the onerous conditions imposed. Mr. Manton.

The use of a bucket-and-ladder dredge, with buckets of, say, 40 cu. ft. capacity, with possibly one "horn", in hard ground, to every two ladder-buckets, doubtless would have involved the breaking up of the larger boulders, say, from 10 to 20 tons, subsequent to the passing of the dredge; but this might be considered as an advantage, for it would keep the dredge at work for a greater proportion of the 24 hours. Mr. Rosewater has stated that boulders were brought up—sometimes twenty times daily—so large as to stick in the bucket, or to necessitate blasting them in the scow to enable them to pass through the doors. Dealing with large boulders with such a machine as a dipper-dredge would seem to delay unduly the dredge, scows, and tugs, and, in addition, to increase seriously the repairs and maintenance costs.

With the bucket-and-ladder dredge, the smaller buckets would work around such boulders, ultimately tumbling them into the bottom of the cut, to be lifted subsequently with a crane, or blown up to be re-dredged.

There may be some novel points in this discussion, which may elicit a reply as to the reasons for making these dredges of this particular design. They were provided with buckets of two sizes; if 15 cu. yd. was the best size for soft excavation, would it not appear that the 10-cu. yd. bucket for hard and soft rock would be a somewhat large, and therefore expensive, size, involving these excessive stresses and undue operating delays?

A. W. Robinson,* M. Am. Soc. C. E. (by letter).—The dipper-dredges described by Mr. Berdeau are probably the most important examples of this type yet used on large work. The dipper-dredge is especially suited to canals and narrow channels, because it takes up little room, and does not require anchorage lines which would obstruct navigation, as in the case of the ladder-dredge, nor does it move its hull at all when working. These dredges were the main reliance in reducing the slides which involved enormous volumes of excavation during the first years of traffic on the Panama Canal. The work done shows the great capability of this type, each of the three dredges having a demonstrated capacity of more than 3 000 000 cu. yd. per year. Mr. Robinson.

The work of the third machine—ordered after some experience gained with the first two—naturally shows improvement of detail,

* Montreal, Que., Canada.

Mr. Robinson.

which is reflected in the better monthly average output. It is to be regretted that further and more radical improvements were not made in order to reduce the abnormally high cost of maintenance which shows in all three examples, which, if eliminated, would have permitted still larger outputs, and at much less cost.

The cost of maintenance, normally, should be only a small fraction of the cost of operation, but the figures given show that the "maintenance" of the *Gamboa* was 1.5 times the cost of operation, that of the *Paraiso* being 1.7 times, and that of the *Cascadas* 1.4 times. "Maintenance" is stated in the paper to be the "cost of keeping the equipment in first-class condition, and depreciation only." In other words, repairs and depreciation.

The cost of "maintenance" of the *Gamboa* was \$131 350 for 1915, and that of the *Paraiso* was \$125 000. The *Cascadas* was no better, but rather worse, despite the improvement in design, being \$108 111 for 9 months of 1916, or at the rate of \$144 000 for the year. These are heavy figures, and, after making all due allowance for hard driving and conditions of work, they indicate the existence of serious defects of design. As intimated in the paper, the principal defects are to be found in the main hoisting ropes, the dipper arms, and spuds.

The life of the main hoisting rope is stated to be from 3 to 35 days. The replacement of a steel rope, 3¼ in. in diameter and 275 ft. long, at such frequent intervals is a heavy tax, to say nothing of the loss by delay. The first dipper-dredge to be fitted with direct wire rope hoist was built by the Bucyrus Company in 1890, and of this the dredges under discussion are the lineal descendants.

As compared with the old 3-part chain hoist then in use, the direct hoist with single rope was an instant success, and has been used widely. For modern heavy loads, however, the single rope is quite unsuitable, and the writer has developed a system of multiple ropes, consisting of two or more ropes in parallel, by which all the advantages of speed and simplicity of the direct single rope are retained, with great durability.

The importance of the subject of wire rope hoisting for heavy loads at speed merits further remark.

It is commonly supposed that, for good results, the sheave diameter should bear a certain ratio to the rope diameter, say 30 diameters or more. A leading maker of wire ropes lists the minimum size of drum and sheaves in a table which for a 6-strand, 19-wire, cast-steel rope, 1 in. in diameter, is 4 ft., and for a rope 2 in. in diameter is 8 ft. It is quite erroneous to proportion sheave diameters to rope diameters in direct ratio. According to this, assuming that a rope 1 in. in diameter would work well over a sheave 30 in. in diameter (which is true in practice if the rope is flexible and the load and speed are moderate), a rope 3¼ in. in diameter would require a sheave 30 by 3¼ in., or, say, 8 ft., which is what we find in the dredges under discussion, and is mani-

festly too small. It is clear that the resistance to bending of a member is not a linear function of the depth. In solid round members the moment of inertia varies as the fourth power of the depth or thickness. In a wire rope in which the individual wires compose the section, the problem is more complex, owing to friction between the wires under varying tension. In practice, the writer finds that the resistance to bending is proportional to the square of the diameter of the rope, and that sheave diameters proportioned accordingly give good results. Thus, if a rope 1 in. in diameter requires a 30-in. sheave, a 2-in. rope should have a sheave 22 by 30, or 120 in. in diameter, and a 3¼-in. rope would require a sheave about 26 ft. in diameter, instead of 8 ft. as used. Not only is the sheave diameter entirely too small, but the rope in question is subject to eight bendings per lift in passing over the two sheaves, or twelve bendings over three sheaves, not counting the wind in the drum. This is equal to from 700 to 1 000 bendings per hour. It is also subjected to severe twisting on the vertical axis as the boom revolves, and, as this twisting and untwisting takes place in the short distance of 13 ft., it is most injurious to a rope of this size. It is hardly necessary to say that such a rope, used in this way, violates mechanical principles, and must necessarily be short lived, no matter what kind of "extra flexible" or what grade of steel is used.

Mr. Robinson.

Another common error is to assume the same diameter for drum and sheaves, as in the maker's catalogue referred to. Drums may be considerably smaller than sheaves, for equal wear. This is evident when we consider that the rope makes two bendings in passing over a sheave, once on and once off, and only one bending in winding on a drum. The wear from bending, therefore, is only half as much on a drum as on a sheave of equal size.

The dipper arm used on these dredges is of the split type, and about 40 tons in weight. The use of the split type of arm with rack and pinion is a survival from early and small dipper-dredges for light and shallow work, and, in the writer's opinion, is quite unsuitable when applied to modern heavy dredges for deep water. The concentration of load due to the weight alone on the narrow shroudings of the rack is excessive, and the load and wear on the teeth, when pinched, due to side motion and general stress and twisting, is quite beyond the safe limit of endurance. Furthermore, the lateral strength and resistance to twisting of the split type of arm are small, and the construction is such that, when over-stressed, permanent deformation takes place. Consequently, the wear and tear is severe, and the cost and delay due to the renewal of these parts is a large item. No amount of increase of section of this design will remedy the trouble; on the contrary, it will aggravate it. The great weight of the dipper arm is a predisposing cause of wear, and likewise is very severe on the dipper when dropped on the bottom.

Mr. Robinson.

The question of spuds for heavy stresses in deep water is an important one, and here, again, the methods of early days fail, and an entirely new treatment of the subject is demanded. For many years, and for depths, on the Great Lakes, a plain wooden spud sliding in plain wooden vertical guides answered every purpose. As depths and stresses increased, steel spuds were tried, but failed in many cases owing to lack of resilience and also to the concentration of the load at one point. The ordinary methods of construction become inadequate and unsuitable when we have to deal with the enormous stresses generated by machines of great power working in hard material and in deep water. The pressure of a square spud against the sides of a parallel slide in which it fits more or less loosely is nil along the middle portion of the slide and is concentrated at the top and bottom; this results in shearing the rivets at points of concentration, and often causes deformation and rupture. Furthermore, a steel spud being a rigid member in a more or less rigid casing, there is no elasticity, as in a wooden spud. Strengthening the parts and making the spud of heavier section does not remedy the trouble; on the contrary, with an enormously heavy spud, the difficulty of handling it is increased, and the rivets loosen up as before, unless suitable methods are adopted to distribute the stress over a sufficient number of rivets and provide the necessary elasticity.

The author states that these spuds have to be removed every 90 days, three having been broken in as many days in one case.

The stresses to which they are subject cannot be closely calculated, as in a bridge, but no bridge designer would think of subjecting a riveted box-beam, of the dimensions stated, to a bending stress of 150 tons or more concentrated on a few rivets, such a beam also bearing, as a column, a heavy vertical load.

In addition to the foregoing points of criticism, to which the author has drawn attention, the following additional points may be mentioned.

Method of Stiffening Hull.—The steel hull is stiffened by two fore-and-aft steel trusses. This method is a survival from the days of wooden hulls, which required stiffening. The writer thinks that better stiffness, with less weight and obstruction of space, can be provided by a suitably designed steel hull, without trusses, but which possesses the necessary strength in itself. The bottom of the hull, the steel main deck, and the top member of the truss, together constitute a beam with three flanges, which is an absurdity. Some excuse for the truss is found in the support it gives to the overhead turn-table, but this would be much better on deck, where it would be on a solid foundation, and would permit the operator to have an unobstructed view of his work, a defect to which the paper calls attention.

Main Engines.—The main engines are compound, with outside cranks. The writer thinks that simple engines, with center cranks,

would be preferable. Compounding an engine which stops every 40 sec., runs at full load about 10% of the time, and at all sorts of speeds, does not result in any economy. In locomotives, which run more uniformly, compounding has gone out of use principally on account of the value of simplicity and good starting power—both essentials on a dipper-dredge—and because the economy was not sufficiently marked to make it worth while. Some economy might be had by adopting higher pressures and by superheating. The side cranks, again, are a relic from small hoisting engines used in early days, and are out of place in marine practice, or in large engines subject to sudden changes of speed and load. Mr. Robinson.

Boom.—The boom is of the curved bowstring design, which the writer used for many years, but found deficient in lateral stiffness against the side thrust of the dipper arm, and weak at the lower end, due to its shallow depth at that place and its inability to resist twisting stresses. Therefore, he designed a boom with straight tapering members, having wide bottom flanges and great depth at the lower end; this provided freedom of movement to roll, instead of attempting to resist rigidly the stresses due to swinging. This boom is sometimes known as the "Atlantic type"; it possesses great strength, with less weight and fewer parts.

Inaccessibility of Parts.—The author has already commented on the difficulty of dismounting the brake wheels and stripper shaft, due to design. The writer would also criticize the arrangement of having so much steam machinery below deck in a tropical climate, some of it, such as the spud engines behind the trusses, in an inaccessible position.

Capacity of Dipper.—Though called 15-cu. yd. dredges, it does not appear that they have ever been worked with a dipper of that size, even in soft material, and 10 cu. yd. seems to be the useful limit of capacity, although the dimensions of the dipper, as stated in the paper ($10\frac{1}{2}$ by 9 by 9 ft. over all), would indicate a somewhat less capacity. With a very large dipper, it is not alone a question of the power and ability of the dredge to handle it, but of the ability of the scow to receive the impact of such a mass of material and dispose of it.

Character of Work.—The paper states that these dredges were "placed in Gaillard Cut in rock digging exclusively." A more specific classification of material dredged would be necessary in order to form a just idea of the work done. Although some rock may have been present, "rock digging exclusively" is misleading. The sliding in of the banks of the canal, and the large output, both prohibit the idea of "rock exclusively."

The defects which have been criticized illustrate a tendency to follow beaten tracks, which lead astray when followed too far. This tendency is found in many branches of engineering, but especially in the design of dredging machinery, the early development of which has been

Mr. Robinson.

largely in the hands of so-called practical men with slight acquaintance with engineering. By a process of strengthening parts that failed or proved weak, successive examples were built up into a system from which it was thought unwise or impossible to depart. In many instances, tons of metal were introduced into the design of parts in this way as the result of experience, and the degree of success resulting was an apparent justification, whereas the truth is that entirely different designs based on engineering principles and sound reasoning would have been better.

The splendid work which these dredges have done, in spite of the mechanical defects noted, is a tribute not only to the excellence and solidity of the machines as a whole, but especially to good operation and management at a time when the passage of the Canal was at stake.

Mr. Rosewater.

WILLIAM M. ROSEWATER,* ASSOC. M. AM. SOC. C. E.—This paper gives a brief description of the machinery used on these dredges, together with some suggestions for changes which, in the author's opinion, would improve future dredges. The performances of these dredges are tabulated, and some figures are given relating to the output and cost of operation, but, unfortunately, the conditions of operation are not described, and, without knowing these conditions, the records of performances cannot be intelligently compared or used in predicting what other dredges, or even the same dredge, may do on other work.

The speaker will try to describe the conditions under which these performances were made, and also explain how the original specifications for these dredges were decided on.

At the time the first large slide closed the Canal to navigation, the Panama Canal Commission was operating a number of hydraulic dredges, a few dipper-dredges equipped with 5-yd. dippers, and the ladder-dredge, *Corozal*, with buckets of about 35 cu. ft. capacity. All these dredges were found to be unsuitable for clearing away this tremendous mass of broken rock and earth. The dipper-dredges were too small and of too light construction, and the *Corozal* was unsuitable for the work because the material had lost all its original regularity of formation, and was no longer in layers of original uniform material, through which one could steadily feed an endless chain of buckets, but was a mass of earth mixed with broken pieces of rock of all sizes and shapes, and lacking all uniformity of formation, a mass against which, on account of this lack of uniformity, no endless chain of buckets, with its uniform speed and original close formation (as compared with the enormous size of the pieces of rock), could be expected to cope successfully.

Such a mass calls rather for a large single bucket or dipper, the direction, point of application, and speed of which are constantly under

* South Milwaukee, Wis.

Mr. Rosewater.

control, and with which the operator can feel his way as he plows through the mass.

Such were the conditions that led to the placing of orders for these large dipper-dredges. A few years previous to this, W. G. Comber, M. Am. Soc. C. E., then in charge of the dredging, had visited the United States to inspect the more important large dipper-dredges on the Atlantic Coast and the Great Lakes. Of all these dredges only two types were considered as coming within the requirements for this work; these were the 15-yd. dipper-dredge, *Toledo,* built by the Bucyrus Company for George H. Breymann and Brothers, of Boston, Mass., and the two 10-yd. dipper-dredges working on the Cape Cod Canal. The latter were sister machines, and were built by the Atlantic Equipment Company, frequently referred to as the American Locomotive Company. After careful inspection and study of these dredges, the officials of the Panama Canal decided that the *Toledo* represented the type most suitable for their work. The *Gamboa* and the *Paraiso* were, with certain modifications, to be duplicates of the *Toledo,* the largest dredge of this type, and practically the only one that met, in a general way, all the requirements of the work on the Canal. This dredge carries a 12 or 15-cu. yd. dipper, depending on the nature of the material to be dredged, and can dig to a depth of 50 ft. below the water line.

The *Toledo* has a wooden hull, with structural steel stiffening trusses, a single boiler, and wooden spuds, the forward spuds being operated by a two-part wire rope tackle. The Panama dredges were to have steel hulls, also structural steel trusses, two boilers, and steel spuds, the forward spuds to be operated by a four-part wire rope tackle. In practically all other respects these dredges were to be duplicates of the *Toledo.* Suggestions for improvements in the design of the main hoisting engine, drums, and gears had been made by the speaker's company at that time, but were rejected by the Canal officials, chiefly on account of the extra time it would have taken to get out an entirely new design. The need for these dredges on the Canal was so great that, in view of the satisfactory work done by the *Toledo,* the Canal officials felt justified in rejecting suggestions for changes in design which if incorporated would have delayed the delivery of the dredges.

When the order for the third dredge, the *Cascadas,* was placed, the first two had been in operation for about 8 months. The work of these dredges had been highly satisfactory, and the only changes requested were that the hull be widened so that it would be flush with the outside of the forward spuds. This increased the width of the hull about 11 ft., and added to its transverse stability in maneuvering the dredge with the spuds in the high position. It also provided space for a third boiler, which was considered desirable. A gallows frame was also to be added at the bow, so as to permit raising the forward spuds from their tops, thereby avoiding the use of sheaves at the bottom of the

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spuds, as in that position they were easily clogged with mud, and were a source of more or less trouble on the older dredges. Changes in the design of the main hoisting engine, drums, and gears were again considered, and rejected as not being worth while, but it was decided to omit the reversing links and add a separate reversible engine, to be thrown into gear, when required, to turn the main drum slowly and thus facilitate the renewal of the cable and when making repairs.

The performance of all these dredges is unique, and the speaker doubts very much that it can ever be equalled elsewhere, even with the same or similar dredges, because all the conditions necessary for continuous maximum output will probably never again be present to the same extent as on the Canal at the time these performances were made.

The conditions necessary for maximum output are as follows:

1.—A depth of cut sufficient to insure a full dipper for every stroke.

2.—A channel free from traffic, thus causing no delays on account of passing vessels.

3.—A channel suitable to permit setting adrift the loaded scows as fast as filled. These were pushed out of the way by the empty scow, and picked up later by a steam tug. This method of handling scows could not be used at all times, of course, because some of the slides completely closed the Canal, and then, naturally, the loaded scow had to be moved out of the way before an empty one could take its place, and, at times, the scows even had to be turned end for end before they could be completely loaded.

4.—No delays caused by waiting for empty scows. This required, not only an ample supply of scows and steam tugs, and a perfect transportation system, but also a dumping place free from rough weather and storms, so as not to interrupt the towing and dumping service.

5.—A bank of earth slowly moving toward the dredge, so that the latter could work for long periods of time without having to stop to move ahead.

6.—Careful and expert management, with experienced crews, and the division of the 24-hour working day into three 8-hour shifts, and not into two 12-hour shifts.

From Tables 1, 2, and 3, the minimum and maximum monthly outputs for these dredges are shown to be:

In 1914 for the	*Gamboa*	108 000 cu. yd.	to 166 000 cu. yd.	
and " "	*Paraiso*	70 000 " "	to 144 000 "	"
In 1915 " "	*Gamboa*	150 000 " "	to 280 000 "	"
and " "	*Paraiso*	134 000 " "	to 313 000 "	"
In 1916 " "	*Gamboa*	129 000 " "	to 321 000 "	"
and " "	*Paraiso*	196 000 " "	to 306 000 "	"
and " "	*Cascadas*	122 000 " "	to 330 000 "	"

If the output, as given in these tables, is averaged for the fiscal years, the *Gamboa* and *Paraiso,* for the first year's operation, each removed 150 000 cu. yd. per month, and for the second year's operation 250 000 cu. yd. per month for each dredge; the average for the *Cascadas* during its 8 months' work in 1916 was 300 000 cu. yd. per month. Mr. Rosewater.

The *Gamboa* and *Paraiso,* at the start, were operated on only two 8-hour shifts per day, and were changed to three 8-hour shifts at about the time the *Cascadas* was placed in commission. The *Cascadas* was always operated on three 8-hour shifts. The increase of 100 000 cu. yd. per month for the second year's work of the *Gamboa* and *Paraiso* is no doubt largely due to the addition of the third shift, which added about 50% more operating time to each month, but the scow service was also improved at this same time, so that it is the combination of these two that really accounts for the increased yardage.

Assuming twenty-six 24-hour working days per month, the average hourly output for both the *Gamboa* and *Paraiso* was 400 cu. yd., and for the *Cascadas* about 480 cu. yd. The *Gamboa* and *Paraiso* were originally equipped with 10-yd. and 15-yd. dippers, for work in rock and earth, respectively, but it was found that the rock and earth were mixed to such an extent that no distinguishing line could be drawn dividing the classes of work, nor was it found practicable to change the dippers with the change in the character of the material, so that these dredges used the 10-yd. dippers constantly, and the 15-yd. dippers were eventually cut down and converted to rock dippers. The *Cascadas* was equipped with a 12-yd. dipper, suitable for both rock and earth work, and a similar change was intended to be made on the other dredges, but the speaker thinks that this had not been done at the time of these performances. Assuming, then, that the *Gamboa* and *Paraiso* were using 10-yd. dippers, and the *Cascadas* a 12-yd. dipper, each of the three dredges was loading at an average of not less than forty dippers an hour, or a dipperful every 1½ min. To make this average output, the dredges had to work very much faster than this, for these figures do not allow for lost operating time; and, to illustrate just how fast it was possible to operate, *The Canal Record* of February 23d, 1916, stated that on February 18th, 1916, in a 24-hour period, from midnight to midnight, the *Cascadas* loaded into scows 23 305 cu. yd. of earth and rock in 23 hours and 15 min. actual working time. This is at the rate of 1 002 cu. yd. per hour, or 16.7 cu. yd. per min., or a dipperful about every 40 sec. This also illustrates the fact that the dredge lost practically no time whatever in doing this work, and that it probably was not even required to change its position, as a shift of position usually causes a delay of from a few minutes to an hour, depending on the location and the character of the ground in which the dredge is working.

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In this same month, the shortest of the year, this dredge appears also to have made its record maximum output of 330 000 cu. yd., an average of 550 cu. yd. per hour, which is approximately 55% of its best single day's record.

The best record for the *Toledo,* so far as the speaker knows, has been 9 000 cu. yd. in 15 hours, and her monthly output varies from 100 000 to 200 000 cu. yd., but the conditions under which most of her work was done are widely different from those existing at the Panama Canal. The depth of cut generally was shallow, from 6 to 10 ft., and the time lost in moving the dredge ahead was generally very appreciable, but the largest loss of time is usually caused by rough seas, which prevent taking the scows out to the dump. At some seasons of the year this loss amounts to practically two-thirds of the available working time.

From the foregoing it will be readily seen to what an extent the conditions at Panama favor the work and affect the yardage obtained. For efficient results, not only must the dredge be of proper strength and design, and suitable for the work, but the transportation problem, also, must be very carefully solved, in order to take full advantage of the possibilities of the dredge.

Tables 1, 2, and 3 also give the cost of operation per cubic yard, and show that it was remarkably low. These figures, however, cover only the cost of loading the material into scows, and do not include the towing and dumping. As dredging contracts, as a rule, include towing and dumping, the speaker would suggest that Mr. Berdeau add, if possible, a supplement showing the cost per cubic yard of spoil on the dump, including all overhead charges, and also giving the length of tow per round trip. From figures pertaining to this work which the speaker has seen, the cost of towing will add approximately 20 cents per cu. yd., and will show that the cost of spoil on the dump varies from 24 to 33 cents per cu. yd. These figures cover the performance of the *Gamboa* and *Paraiso* from January to October, 1915. The length of the tow per round trip was about 25 miles. For the *Cascadas,* on account of its slightly larger average output, the cost figures ought to be slightly less than those given.

Some of the details will now be taken up in the order in which they appear in the paper. Mr. Berdeau submitted to the speaker's company a copy of his paper for correction before it was printed, but it is found that several slight errors were apparently overlooked, and, for the sake of accuracy, the speaker will try to correct them.

On page 516, the acceptance of the *Cascadas* is given at Port Richmond, N. Y. This is an error. The *Gamboa* and *Paraiso* were completed and accepted at Port Richmond, N. Y., but the hull of the *Cascadas* was built by the New York Shipbuilding Company at Camden, N. J., where the Bucyrus Company put in the machinery and completed the dredge.

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The next correction is in regard to the depth of the *Gamboa* and *Paraiso*. The hulls of these dredges are 16 ft. 6 in. deep at the bow and 13 ft. 6 in. at the stern.

The dippers or buckets, as supplied by the Bucyrus Company, were of stronger and heavier design than any that had ever been built before. Each 10-yd. rock dipper weighed nearly 20 tons, and each 15-yd. dipper, 21 tons. The rock dippers had been in service only a comparatively short time when they began to show signs of distress, and had to be overhauled. An attempt was made to hold the Bucyrus Company responsible for the failure of these dippers, on the ground that this failure showed they were too light and inadequate for the work. The speaker's company, however, had no difficulty in presenting proof to F. C. Boggs, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. A., then Purchasing Officer for the Panama Canal, that these dippers were stronger and heavier than any then in service, and that the company had faithfully and to the best of its ability fulfilled its contract.

These dippers are attached to a handle weighing approximately 37½ tons, and, together with the dipper itself, there is a total weight of 57½ tons in motion, and if, in lowering, the operator misjudges the distance, he can easily allow the dipper to strike bottom with sufficient momentum to be crushed. The speaker's company quite naturally concluded, from the first meager reports cabled to it, that this had been the cause of the trouble, just as Mr. Berdeau has done in his paper. However, the detailed reports, when received, showed that the parts had been forced outward and not inward, as would have been the case if the destructive force at work had been due to the weight of these parts; and, finally, when all the facts became known, it developed that these dippers frequently brought up large pieces of rock, too large to go through the dipper and also too large to pass through the openings in the dump scows. Sometimes these pieces were lodged on top of the dipper and at other times they were wedged tightly in place, and as they were too large to pass through the dump scows, they had to be broken up. This was done by placing a light charge of dynamite on top of the rock, covering it with clay, and then lowering the dipper below the water surface, but still allowing it to hang on the hoisting rope when the charge was exploded. This happened sometimes as often as twenty times in a working day, and it not only broke up the rock, but also eventually the dipper. This naturally had not been contemplated in designing these dippers, and it fully accounted for the manner in which the parts failed. It is not at all surprising, therefore, that these dippers had to be overhauled and repaired every 3 or 4 weeks. This treatment of the rock was also responsible for

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some of the short-lived hoisting cables. Inspection of the cables showed that blasting the rock in the dipper broke a few wires of the cable each time, and naturally shortened its life.

Cables.—The life of the cable in ordinary service is usually represented by 300 000 cu. yd. The life of the cables at Panama, in view of the continuous high-speed operation of the dredges, and the treatment that was sometimes given these cables, does not seem excessively short.

The suggestion has been made that sheaves of larger diameter would increase the life of the cables, but the speaker doubts that this would result in any material benefit, because the drum diameters are considerably less than those of the sheaves, and a cable usually gives way at the point where the radius of bend is the smallest.

Dipper Handles.—It was specified that the dipper handles should be duplicates of the one on the *Toledo,* but more heavily armored; the handle of the *Toledo* had originally been specified to be a duplicate of one used on an older and smaller dredge, so as to make one spare handle serve for either dredge. In this way the width of 12 in. for each dipper stick was determined arbitrarily, and was incorporated in the design of the Panama dredges. The speaker has always been of the opinion that a stick 16 in. wide would be far better for dredges of this size, but a change in the width of these sticks would entail many other changes which would affect the initial cost of the dredge quite materially, so that, in the end, the adoption or rejection of the suggested improvement usually rests with the purchaser, and not with the Engineering Department.

Saddle Blocks.—The saddle blocks, as stated by Mr. Berdeau, are of the latest and most approved type, and have given general satisfaction. The block is on the outside of the handle sticks, instead of between them, and is provided with adjustable caps for holding the handle rack in proper mesh with its pinion. It is essential to keep the saddle block in proper adjustment as the handle wears, for the forces at work tend to separate the rack from its pinion, and, if the clearance is allowed to become too great, this will cause the teeth to jam and break, and even break the saddle block itself. Rollers in place of slide-plates would undoubtedly reduce the wear at this point, and Mr. Berdeau's suggestion for their use is a good one, but, before it can be adopted, the old and long established prejudice against rollers for this service will have to be overcome. Some 15 or 20 years ago, rollers were considered standard practice, but the design was inadequate, and caused these roller slide-plates to be condemned.

The use of the single dipper stick with a divided hoisting rope—one rope on each side of the stick—has been suggested from time to time. The difficulty with this is that no one has yet found a

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way to lead the ropes from the widely separated sheaves at the foot of the boom through the turn-table to the sheaves in the hold at the bow of the dredge, and still retain the proper lead of the rope for all positions of the boom. Nor has any arrangement yet been perfected that would permit leading these cables directly from the point of the boom to the drum without going through the turn-table. Such an arrangement is often used on Lidgerwood hoists operating clam-shell buckets, but cannot be used on a dipper-dredge because of the interference of the dipper handle with these cables as the boom swings toward the corner of the dredge.

On some of the large revolving shovels built by the Bucyrus Company the two members of the dipper handle have been separated far enough to allow the entire boom to be placed between them. This permits the use of a box-girder boom having maximum strength and stiffness for any given weight of material. It also increases the torsional resistance of the handle on account of the wide spread of its members, but this design is not well adapted for use on deep-digging dipper-dredges because of the interference in certain positions of the boom of the handle with the A-frame legs. This might be overcome by using A-frames with bent or bowed legs, but such frames would have to be of extremely heavy construction in order to take care of the added bending moment. The widely separated dipper sticks would also require a broad section of dead wood below the boom, which would add greatly to the resistance in pulling the dipper back with the backing cable, especially if working in a strong current; and a further objection might be raised to this that the current would tend to break the dipper sticks by forcing them against the bottom corner of the hull.

Booms.—The booms are alike on all three dredges, and are of plate-girder construction, being 25% heavier than the one on the *Toledo,* which is of the latticed-girder type. The boom, without doubt, is the most difficult part of the dredge to design. It is subject to extremely heavy loads, some of which are indeterminate; its shape and the location of some of its principal members are very often fixed by the physical requirements, so that the result is at best more or less of a compromise between what one would like to do and what one can actually do. By indeterminate loads the speaker means those caused by starting to swing the boom before the dipper is clear of the water, and sometimes before it is clear of the bank, also those caused by the sudden stoppage and reversal of swing necessary when working at the highest speed. Then, too, it is not uncommon practice to move the dredge sidewise by swinging the boom with the dipper bedded on the bottom. All these forces produce reversal of stresses, and have a far-reaching effect that sooner or later makes it necessary to overhaul the boom, no

Mr. Rosewater. matter how well it was designed and built. It has been the speaker's experience that the heavier and stronger the boom is built, the more likely it is to be overworked; in other words, when the boom looks extra strong and heavy, the operators do not hesitate to do with it what they would be afraid to undertake with some of the lighter booms that are in successful operation.

Main Engines.—The main engines are of the twin, tandem, compound type. In a 40 or 50-sec. cycle of operation, these engines run only for 10, 12, or 15 sec., at most, and are idle for the remainder of the time. From this, the speaker believes, it will be readily perceived that the conditions can hardly be right for the use of a compound engine, and for this reason the speaker has always advised against their use, but, as these dredges show, without success. The compound engines were insisted upon for the *Toledo,* and thus they came to be used on the Panama dredges.

The speaker is frank to confess that, as far as he knows, no indicator cards have ever been taken from any of these compound engines, so that he does not know what the low-pressure cylinder is actually doing. Mr. Macfarlane, the Superintendent of Dredging on the Canal, stated a year ago that, if he were to order a fourth dredge, he would want it provided with simple, double-cylinder engines, and not compound.

The statement in the paper that the low-pressure cylinders have piston valves is in error. It is the high-pressure cylinders that have the piston valves, the low-pressure cylinders have double-ported slide-valves, with American balanced slide-valve disks.

Hoisting Drum and Gear.—The drum is of the so-called differential type, that is, it is of two diameters, the connection between the small and large diameter being made by two or three turns of a grooved spiral section. The original idea of this, as explained in the paper, was to obtain a slow speed and maximum digging effort when the dipper was on the bottom and a fast speed with proportionately reduced effort while the loaded dipper was being hoisted.

The small part of the drum is usually of a diameter that is rather small for the size of the cable, and its length, proportioned for average conditions, cannot possibly be right for all conditions. Thus, on a deep-digging dredge, unless the cable is especially shortened to suit the job in hand, most of the work in a shallow cut will be done on the spiral and large diameter parts of the drum, and the dredge will thus be at a disadvantage for power. This would also be true for a deep cut in any soil in which the dipper must cut its way clear to the surface of the water. Trouble has also been experienced with drums of this type on the older dredges, which are now working with reduced boiler pressure, in the event that the operator misjudges his clearance height and stops hoist-

ing before the dipper is high enough to pass the hatch combing of the scow, for then, if he is swinging fast, he cannot stop, nor can he get the hoisting engine to start the loaded dipper from rest in that position without first lowering it to obtain a more favorable starting position, and, when there is not time for this, a collision and damage to both scow and dredge is inevitable. Such accidents, however, do not happen where the original boiler pressure is carried, for the speaker always aims to have sufficient hoisting power to start the dipper from any position. Mr. Rosewater.

To overcome these objections, of late years, the speaker has been recommending the straight drum of large diameter, and with gears properly proportioned to give the correct digging effort and speed. With a drum of this type the available power will not be affected materially by the depth of digging, and the engine can be readily designed so as to accelerate rapidly and run at higher speed the moment the dipper finishes its cut and the comparatively easy work of hoisting the load begins.

One of the chief obstacles to the straight drum of large diameter has been its increased initial cost. The larger drum calls for more powerful friction clutches and brakes, and requires also heavier and more powerful gearing. This, translated into commercial terms, means increased initial cost, and usually resolves itself into a question that the purchaser decides without much reference to the engineering points involved.

The use of twin cables running side by side has been advocated by many, and tried on a number of large dredges, as a solution of the hoisting cable question; in fact, twin cables were suggested for use on the *Cascadas*. It was rejected by the Government engineers, after careful consideration. The chief argument against the use of the twin cables is that it is impossible to maintain them at exactly the same length. They will stretch differently, and there will be slight differences in the off lead from the sheaves to the drum which will also tend to vary their length, so that an efficient equalizing arrangement is necessary, and, with the machinery as arranged in most dredges, the only place for this equalizer is at the dipper bail, where the space available will not permit the use of sheaves of large diameter; with sheaves of small diameter, the life of the twin cables has been no greater than that of the single cable, and the cost of the twin cables has been greater.

Swinging Circle and its Machinery.—The swinging circle and its machinery are of the usual type constructed by the Bucyrus Company. The division of each swinging rope into two parts, to facilitate its renewal, and to conserve incidentally the portion that receives very little wear, is so simple an improvement that it seems remarkable that

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no one had ever thought of it before. The reason for this, no doubt, is that ordinarily these cables are changed so infrequently and that there is usually plenty of spare time to make this change while the dredge is lying idle, that no one ever thought of making an improvement here until the conditions changed as they did at Panama.

The mechanism for controlling the throttle-valve and reversing the swinging engines with a single lever is new. On the older and smaller dredges this was effected by merely connecting the links and the throttle to the same lever system, but, on the larger dredges, the links require too much power to permit them to be operated in this manner. On these dredges, therefore, the links are reversed by a steam cylinder the direction of motion of which is controlled by a special slide-valve. The arrangement used connects this controlling slide-valve with the lever system controlling the throttle-valve in such a way that, starting from the central or neutral position of the operating handle, the movement of the handle in either direction causes the links, during the first portion of the lever travel, to be placed in correct position corresponding to the direction in which the handle has been moved, and further movement of the handle then no longer affects the position of the links, but serves only to regulate the throttle-valve. This arrangement has been highly successful, and provides a safe and practically fool-proof method for controlling both the direction and the speed of swinging by using a single hand lever. The only error the operator can make is to move his lever the wrong way thereby starting the boom to swing in the opposite direction from that intended, but, as the lever requires no power for its operation, he can usually discover his error and correct it before any serious damage is done. A similar mechanism is also used for controlling the direction and speed of travel of the forward spuds.

Backing Engine.—Mr. Berdeau states that the diameter of the backing drum is too small for its rope. This has been the case in practically all dredges, and is caused by the fact that, originally, a chain was used for this purpose, and was still retained even after the hoisting, swinging, and spud handling chains were discarded for wire cables. The substitution of a cable for the backing chain, therefore, is only of comparatively recent date. When this change was made on the existing dredge, it was usually effected by salvaging the better part of a worn-out hoisting or swinging cable, and this could easily be done, as the backing cable is only 90 or 100 ft. long. This cable was then used on the drum which had previously wound the chain, and in this way the extremely small ratio of drum to cable diameters was established, and, as the results were comparatively satisfactory, especially where old pieces of other cables were used, there was no incentive to change the ratio, and, even on new dredges, this small ratio was retained when the purchaser discovered that the use of a larger drum,

with its larger gears, clutches, and brakes, added a few hundred dollars to the initial cost of the machinery. Mr. Rosewater.

Forward Spud Machinery.—The chief improvement in the forward spud machinery consists in the use of four-part, in place of two-part, blocks, for handling the spuds. The necessity for this was brought about by the demand in recent years for dredges having booms which could swing through a total of 180 degrees. On the older dredges, the maximum swing seldom exceeded 120°, and with the increase to 180° in the arc of swing it was soon discovered that greater loads were placed on the forward spuds, so that, to permit the dredge to dig with the boom swung to its limit, four parts of cable were required to carry the load.

On the older dredges, using only two parts of cable on the forward spuds, it was a comparatively simple matter to renew the cable, or even to change spuds, so that no one felt warranted in incurring the additional expense of providing a gallows frame for handling these spuds from their tops. When the four-part spud hoists were tried out on the Panama dredges, however, it was soon discovered that as the reaving of a new rope and the changing of spuds was very slow and difficult work, the use of an overhead gallows frame would be well worth its additional cost, and, for this reason, it was adopted on the *Cascadas.*

Mr. Berdeau points out that they are experiencing some trouble with the stern spud machinery because of the wear of the saddle-block parts that hold this spud in proper relation to the pinion with which it is hoisted and lowered. This trouble no doubt could be overcome by improving the design of the saddle block and substituting rollers for the slide-plates. If wire ropes were used for handling this spud, the service of a saddle block could be dispensed with, but wire ropes have not proved satisfactory for this purpose on deep-digging dredges. The chief objection to their use has been that they work so smoothly and noiselessly that the operator allows the spud to fall altogether too fast, and when he attempts to check its speed it frequently has gained enough momentum to break the rope or to tear out the connections for the rope, and, when this happens, the services of a diver are necessary to make repairs. Where the rack and pinion are used for handling this spud, the work is not done so smoothly and quietly, the result being that the noise usually serves to prevent the operator from letting this spud travel too fast.

Deck Fittings.—The two three-drum winches used for moving scows are a very important part of the deck equipment, and it was through their use that the dredges were supplied with a substantially continuous string of scows. In addition to these deck winches, there were also added two steam capstans, near the stern of the dredge, which were used for bringing empty scows up alongside and within reach of the lines from the other winch. These capstans were also useful when it

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was necessary to turn the scows end for end, in order to load both ends of the scow.

Boiler and Fittings.—Special care was taken in the design of the steam piping and the fittings, so as to have everything as nearly perfect and complete as possible. The boilers burn crude oil, using the Dahl patent mechanical oil burners, which burn the oil under pressure, so that no steam is required for atomizing it.

House and Cabin Construction.—Everything in connection with the house and cabins was given a great deal of thought and attention, and all plans were forwarded to the officials at Panama for approval, so as to insure their serviceability for the climatic conditions on the Canal, and also to meet correctly the requirements for the "gold" and "silver" crew quarters.

Hull.—The decision to widen the hull for the *Cascadas* was brought about by the fact that, in maneuvering the other dredges, it was essential to keep the boom pointing straight ahead, especially if all the spuds had been raised high in the air. The operating levers are properly labeled to show what each controls, but, even so, in the haste with which work is being done, there is always the possibility that the operator may pull the wrong lever at the wrong time, and if, under such conditions, he should happen to swing the boom clear to the side, it might be possible to overturn the dredge. On the first two dredges the spuds are outside of the hull proper and project about 5½ ft. on each side. It was decided, therefore, to widen the hull by just this amount. This enabled the builders to provide space for the third boiler, and has resulted in a dredge that has sufficient transverse stability to withstand safely the swinging of the boom to the corner, even when the spuds are in their highest position. It may be added that this apparent lack of stability on the earlier dredges and also on the *Toledo* was well understood by the builders and the operators, and that it is customary to take the necessary precautions to prevent an accident of this kind. In the past, the objection to using a wider hull has been based on the size of the locks of the Welland Canal. These will not pass anything wider than 44 ft., and, consequently, for dredges built in America, there is usually a clause in the specifications to the effect that the dredge may be stripped down to this width.

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MANHATTAN ELEVATED RAILWAY IMPROVEMENTS*

BY F. W. GARDINER AND S. JOHANNESSON,
MEMBERS, AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. CLARENCE E. CARPENTER, GEORGE H. PEGRAM, T. KENNARD THOMSON, EDWARD WEGMANN, AND HOWARD CONSTABLE.

SYNOPSIS

The work under the Manhattan Elevated Railway Improvements was authorized in 1913, and involved in general the addition of a single continuous express track, with express stations, to the Second, Third, and Ninth Avenue elevated railway lines in the City of New York, operated by the Interborough Rapid Transit Company. The work included the building of 23 miles of single-track elevated structure, the erection of about 50 000 tons of steel, the building of 638 foundations, and the construction or reconstruction of 40 stations, most of the work being in city streets often congested with traffic. The traffic on the elevated railway lines was maintained according to the regular schedule throughout the period of reconstruction.

After giving a short history of elevated railroad construction in New York City, the paper describes first the design of the steel structure, the details of which were worked out, keeping in mind the necessity of making possible the erection without interruption of

* Presented at the meeting of February 6th, 1918.

traffic. Next is described the foundation work, which not only included the building of foundations in new locations, but also the removal and reconstruction of foundations under the existing structure carrying traffic. The method of erection is next described, involving in several cases partial or complete removal of the existing structure and replacing it with a new structure, and at all times maintaining traffic. The work also involved, at certain places, raising the existing structure and moving it sidewise, while trains were being operated, and the replacement of a three-span bridge across the Harlem River with a new bridge. The paper also describes the reconstruction of the stations, including the two standard types of express stations used: the "hump" type, which required the center track to be raised above the level of the local tracks so that the platforms for serving the express track could be placed above the local tracks, and the "mezzanine" type, which had two island platforms and a mezzanine station under the level of the tracks. The paper further describes the plant, the method of selecting the contractors, and the cost of the work; and, finally, an account is given of the accidents that happened during the work.

History of the Manhattan Railway Company's System.

Elevated railroad construction in New York City was commenced on July 1st, 1867, when "The West Side and Yonkers Railway Company" started to erect an experimental section on Greenwich Street between Battery Place and Cortlandt Street. This section was completed on July 2d, 1868, and was a single-track railroad. The line was extended as a single-track road through Greenwich Street and Ninth Avenue to 30th Street during the following 2 years, and was opened for traffic on February 14th, 1870. The structure was on the east side of the street through Greenwich Street and on the west side through Ninth Avenue. The trains were operated by an endless chain, driven by stationary engines below the surface of the street, at Cortlandt, Franklin, Bank, and Twenty-second Streets.

This method of traction did not prove successful, and traffic was abandoned in November, 1870, but was resumed again on April 20th, 1871, using a dummy engine, the "Pioneer", weighing 5 tons and drawing three cars.

In the fall of 1871 the bondholders of the Company organized under the General Railroad Laws as "The New York Elevated Railroad Company."

In October, 1872, the company issued a circular, in which it was stated:

"* * * We now take and receive passengers at Morris, Dey, Canal, Twelfth, and Twenty-ninth Streets. We run four unique, elegantly finished and furnished cars, made expressly for our road, capable of seating 44 passengers each, and we take no more than can be seated. We are frequently compelled to refuse passengers after our cars are full. We carry about 1 300 daily. * * We believe we are developing what will enhance the value of real estate, solve the problem of quick transit, relieve our over-crowded streets and side-walks, be of great public service, and a successful paying enterprise. * * *"

During the following years, the line was gradually extended northward on Ninth Avenue, in 1873 to 34th Street, in 1875 to 42d Street, and in 1876 to 59th Street. In 1877 a double-track extension was built to South Ferry, and during 1878 a double track was completed and opened for traffic over the whole line. At the same time the rolling stock was increased, consisting, in 1874, of 10 cars and 6 engines and, in 1876, of 21 cars and 15 engines.

The original structure on the east side of Greenwich Street and on the west side of Ninth Avenue between Battery Place and 59th Street was torn down in sections and replaced by a new structure, which was completed in May, 1880.

In October, 1877, the New York Elevated Railroad Company commenced the construction of the Third Avenue Line, and opened it for traffic on August 26th, 1878, between South Ferry and 42d Street. The line was extended and opened for traffic to 129th Street before the end of the year. The 42d Street branch to Grand Central Station was also opened in August, 1878; the City Hall branch, from Chatham Square to City Hall, was completed in March, 1879, and the 34th Street branch to the East River was opened in July, 1880.

Another company, the Metropolitan Railway Company, formerly called the Gilbert Elevated Railway Company, in the spring of 1876 commenced to construct what is now called the Sixth Avenue Line, between Morris Street and 59th Street, but became involved in legal difficulties, and the road was not opened until June, 1878. The same

company opened the line through 53d Street from Sixth to Ninth Avenues and the line in Ninth and Eighth Avenues from 59th to 155th Streets in 1878.

The Metropolitan Elevated Railway Company also constructed an east-side line from Chatham Square through Division Street, Allen Street, and First Avenue to 19th Street, which was opened in September, 1879; the remainder of the line—which is now the Second Avenue Line—to 129th Street, was opened in 1880.

In May, 1879, the New York Elevated Railroad Company and the Metropolitan Elevated Railway Company were leased to the Manhattan Railway Company (first organized in November, 1875), for 999 years, "in order, by means of one management, to avoid dangers of level crossings and also to perfect the system of Rapid Transit." This was known at the time as the Tripartite Agreement of May 20th, 1879.

The authorized rates of fares at that time were 10 cents for any distance of less than 5 miles, and not to exceed 15 cents for a through passage between the Battery and the Harlem River, except on "Commission Trains", which ran between 5.20 and 7.20 A. M. and between 5 and 7 P. M., when the corresponding rates were 5 and 7 cents. The 5-cent fare was introduced for all hours on all lines on November 1st, 1886. It is interesting to note that as early as January 20th, 1879, the collection of tickets by the conductors was abolished, the passengers on and after that date depositing their tickets in the ticket boxes provided at the exit gates, on leaving the trains. This was changed on June 21st, 1880, to the present method of depositing the tickets in canceling boxes at the entrance.

Another elevated railroad company, the Suburban Rapid Transit Company, was originally chartered in October, 1880, but did not commence construction until November, 1883, when the work on the foundations for the Harlem River Bridge at 129th Street and Second Avenue was started. The road was opened for traffic between 128th and 143d Streets in 1886, extended to 166th Street and Third Avenue in 1887, to 169th Street in 1888, and to 177th Street in 1891. The line from 177th Street to Fordham Road was opened in July, 1901, and from Fordham Road to Bronx Park in May, 1902.

On June 4th, 1891, the Manhattan Railway Company assumed control of the Suburban Rapid Transit Company, and from that date,

therefore, was in possession of all the elevated lines in Manhattan and The Bronx.

During 1899 to 1901, the whole system was equipped for electric traction, and since June 25th, 1903, all trains have been run by electric motive power.

On April 1st, 1903, the Manhattan Railway Company leased the system to the Interborough Rapid Transit Company, which since then has operated the lines.

General Description.

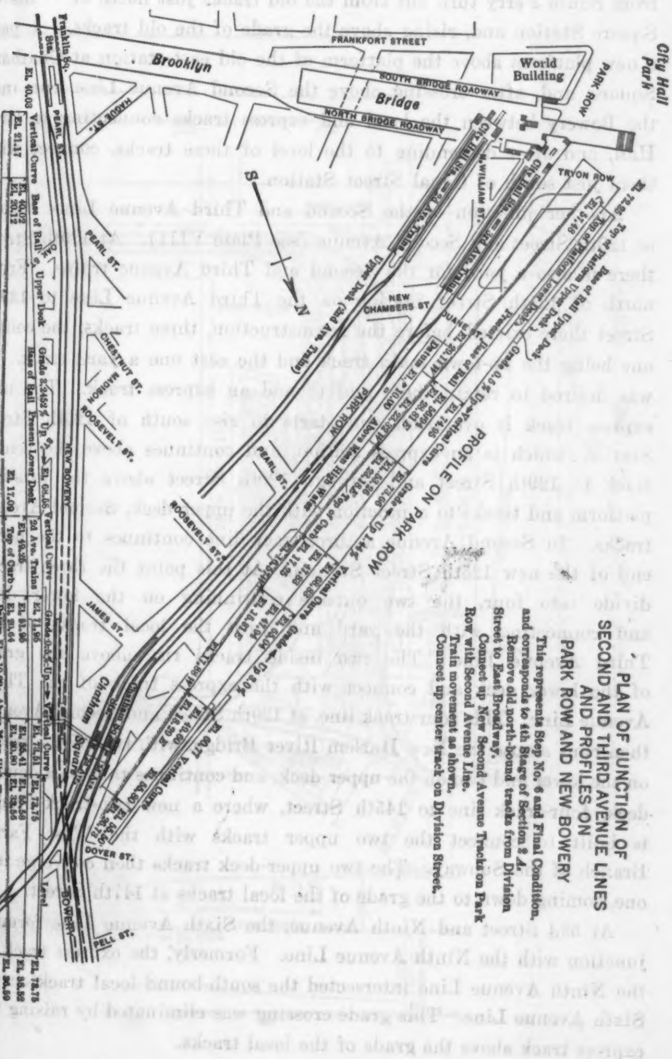
Scope of Work.—The "Manhattan Elevated Improvements" were authorized by a certificate, dated March 19th, 1913, issued by the Public Service Commission of the First District, State of New York, to the Manhattan Railway Company, and formed part of a comprehensive scheme for rapid transit railroads in the City of New York. "Manhattan Elevated Improvements" involved in general the addition of a single continuous express track, with express stations, to the Second, Third, and Ninth Avenue Elevated Lines of the Manhattan Railway Company, operated by the Interborough Rapid Transit Company.

Before these improvements were completed, there existed in certain places on all the lines a center-track construction, and partial express service was in operation on the Third and Ninth Avenue lines; the "Manhattan Elevated Improvements" provided for a continuous express service during the rush hours, down town in the morning and up town in the evening. Two express tracks, of course, would have been better than one, but the cost of providing them would have been prohibitive, as it would have meant actually rebuilding completely the existing lines, and, in addition, it is doubtful if a permit would have been granted for a continuous four-track structure, on account of the objections of property owners. The single express track serves the purposes for which it was intended, namely, to carry the largest number of people speedily to and from their places of business, and to relieve congestion on the existing tracks. The congestion is never due to the operation of trains on the running track, but entirely to the stopping at stations; the more passengers there are to take or leave a train at a platform, the longer the train has to stop at the station; if the number of passengers reaches more than a certain limit, the length

of the stop increases at a much greater ratio than the number of passengers. The single express track relieves the condition by doubling the platform capacity for train service at the points, and at the time, of greatest congestion. The express trains, of course, return on the local tracks, but, as they return empty, in the direction of light traffic, no congestion is produced by them.

Track Changes at Junction Points and Terminals.—As just stated, the general purpose of the work was to provide a single continuous express track in addition to the existing local tracks. This work, however, involved generally at junction points and terminals, extensive track changes, which again necessitated considerable changes in the structure. This applies particularly to the junction points at Chatham Square, at 129th Street and Second Avenue, at 53d Street and Ninth Avenue, and to the terminals at City Hall and at 155th Street and Eighth Avenue.

At Chatham Square, the Second and Third Avenue Lines intersect. Before the reconstruction, the Second Avenue trains continued to South Ferry and the Third Avenue trains either to South Ferry or City Hall. There was a station just north of Chatham Square, in the Bowery, for all Third Avenue trains, and another at the east side of Chatham Square for all South Ferry trains. It was desired, as a part of the reconstruction work, to continue, in addition, the Second Avenue Line to City Hall. The express tracks, both on the Second and Third Avenue Lines, stop just north of Chatham Square, but, nevertheless, the new track layout contains eight tracks through Chatham Square, two of which are overhead in order to avoid the dangers and delays due to grade crossings. The new track layout in this vicinity is shown on Fig. 1. Commencing at City Hall Station, the Third Avenue tracks run on the lower deck past a new island platform at the west side of Chatham Square. Just north of this platform they divide into three tracks—one for the express trains—and continue north through the Bowery. The Second Avenue tracks start on the upper deck at City Hall Station, but come down to the lower grade when reaching Chatham Square, and run past a platform parallel to the Third Avenue City Hall platform into Division Street, where they divide into three tracks—one for the express trains. On the South Ferry Branch, the Second Avenue Line takes the old tracks past the old station at the east side of Chatham Square and connects with the Second Avenue



PLAN OF JUNCTION OF SECOND AND THIRD AVENUE LINES AND PROFILES ON PARK ROW AND NEW BOWERY

This represents Step No. 6 and Final Condition, and corresponds to 4th Stage of Section 2 A.
Remove old north-bound tracks from Division Street to East Broadway.
Connect up New Second Avenue Tracks on Park Row with Second Avenue Line.
Train movement as shown.
Connect up center track on Division Street.

FIG. 1.

tracks from City Hall at Division Street. The Third Avenue tracks from South Ferry turn out from the old tracks just north of Franklin Square Station and, rising above the grade of the old tracks, run past a new platform above the platform of the old east station at Chatham Square, and, after crossing above the Second Avenue Line, run into the Bowery between the local and express tracks connecting to City Hall, and after descending to the level of these tracks, connect with them just south of Canal Street Station.

Another junction of the Second and Third Avenue Lines occurs at 129th Street and Second Avenue (see Plate VIII). At 129th Street there is also a yard for the Second and Third Avenue trains. From north of 125th Street Station on the Third Avenue Line to 129th Street there existed, before the reconstruction, three tracks, the center one being the up-town main track and the east one a yard track. It was desired to retain these and to add an express track. The new express track is overhead, and starts to rise south of 125th Street Station, which is an express station, and continues above the center track to 129th Street and through 129th Street above the existing platform and tracks to a junction with the upper deck, Second Avenue tracks. In Second Avenue a three-track line continues to the north end of the new 125th Street Station. At this point the three tracks divide into four, the two outside continuing on the lower deck and connecting with the yard and with the local tracks of the Third Avenue Line. The two inside tracks rise above the grade of the lower deck and connect with the express track of the Third Avenue Line. This four-track line, at 129th Street and Second Avenue, then runs across the New Harlem River Bridge, with two of the tracks on the lower and two on the upper deck, and continues thus as a double-deck, four-track line to 145th Street, where a new two-track branch is built to connect the two upper tracks with the West Farms Branch of the Subway. The two upper-deck tracks then converge into one, coming down to the grade of the local tracks at 147th Street.

At 53d Street and Ninth Avenue, the Sixth Avenue Line forms a junction with the Ninth Avenue Line. Formerly, the express track on the Ninth Avenue Line intersected the south-bound local track on the Sixth Avenue Line. This grade crossing was eliminated by raising the express track above the grade of the local tracks.

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
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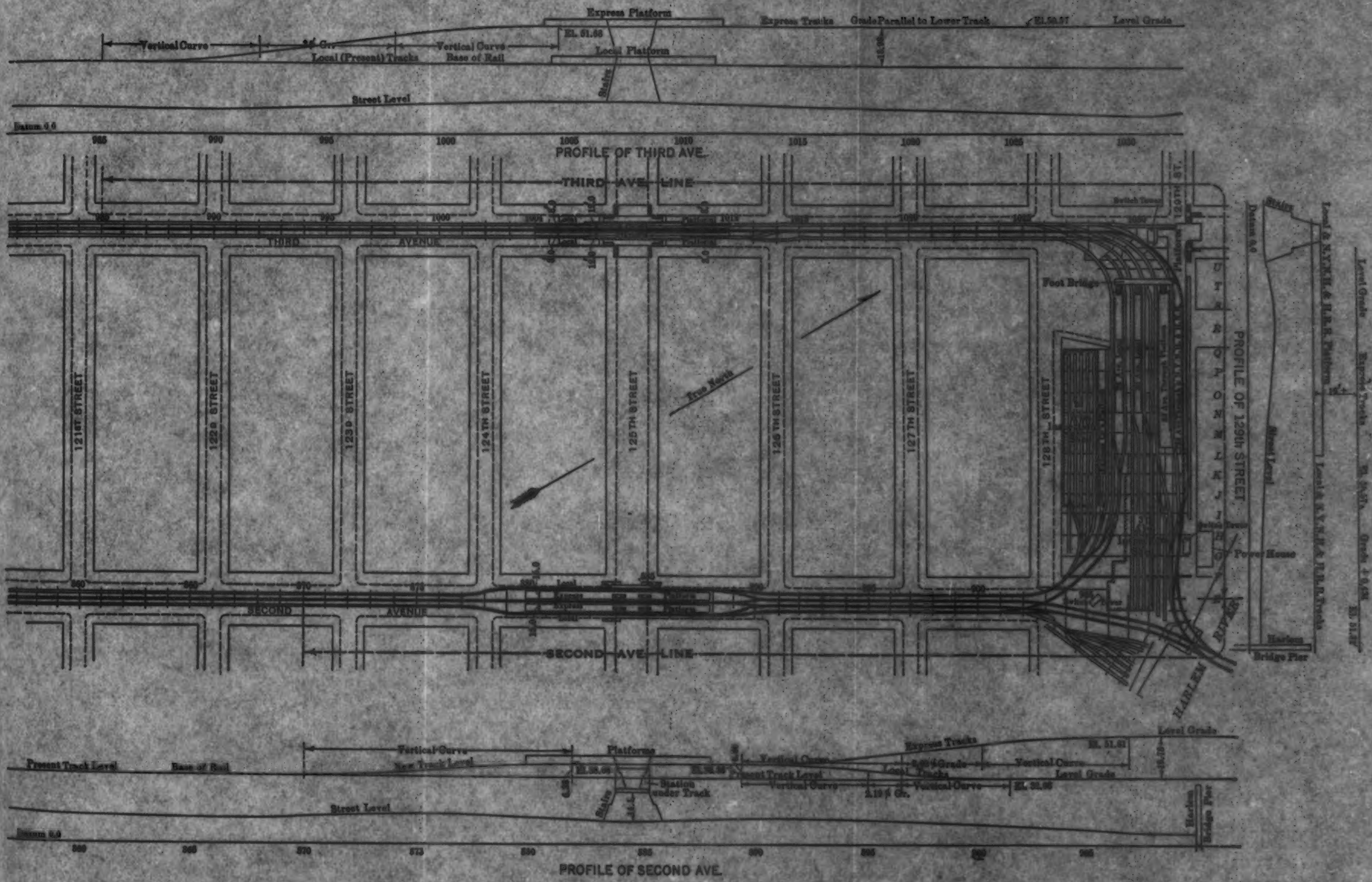
ELEVATED RAILWAY IMPROVEMENTS.



The terminal station at City Hall served formerly only the Third Avenue Line. The terminal facilities consisted of two tracks with a center platform and two side platforms. These facilities were duplicated on an upper level directly above the existing terminal station, to serve the Second Avenue Line, and necessitated practically a complete rebuilding of the structure at this point.

The Ninth Avenue Line terminates at 159th Street, where there is a Company Yard. The terminal station was formerly at 155th Street, and this also formed the terminal of the Putnam Division of the New York Central Railroad. A connection of the Ninth Avenue Line with a new rapid transit line in Jerome Avenue is now being built, and, when it is completed, the terminal of the Putnam Division will be removed to the other side of the Harlem River, and the Elevated Railroad trains will run across the Putnam Bridge. The old station at 155th Street had two side platforms, with adjacent tracks and a center track. North of the station two of the tracks continued to the yard. The new arrangement cannot be completed until the Putnam Terminal is abandoned. There will be, when completed, two new island platforms between 155th and 157th Streets, each platform having two adjacent tracks, and in addition a fifth, which is a yard track, on the west side of the structure to the south end of the station platforms. At present the three westerly tracks and the west platform are completed and in operation. When all the work is completed, the west platform will be used as a terminal platform and the east platform for the through trains. In order to carry the north-bound trains to the terminal platform without a grade crossing, the express track is raised south of the station and the north-bound local track is divided into two branches, one of which runs under the express track to the west side of the structure. The express track is brought down to grade again immediately before reaching the station platforms.

Express Stations.—Although, in a number of cases, local conditions required express stations of special types, two standard types were used whenever the conditions permitted:

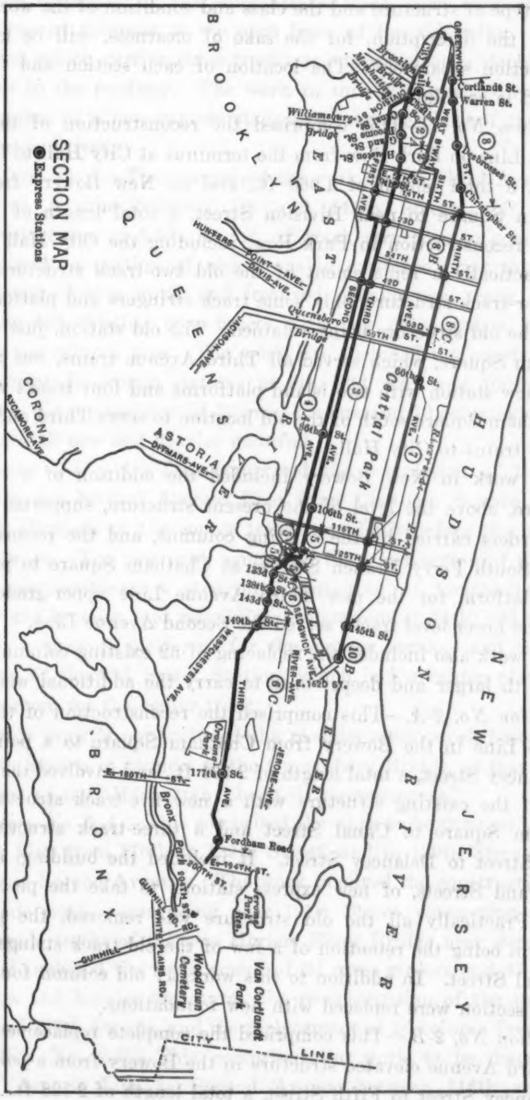
The one type which required entire rebuilding of the existing station, has two island platforms, with the express track between the platforms, and a mezzanine station below the track structure, and was used when the head-room was sufficient to place the mezzanine station under the structure. The tracks, which have a standard spacing of 12

ft., are spread at the location of the station so as to make room for two platforms. The west platform is used for down-town traffic only, the local trains discharging and receiving passengers on the west side of the platform and the express trains on the east side. The up-town express train receives and discharges passengers on the west side of the east platform and the up-town local trains on the east side. Stairs provide connection between the platforms and the mezzanine station, which occupies the space under the track structure between two adjacent bents, and contains the ticket office, waiting-rooms, toilet-rooms, and heater-rooms. Stairs connect the mezzanine station with the street.

The other standard type is the "hump" station, which was used when sufficient head-room could not be obtained to place a mezzanine station under the structure. This type of construction was conceived by George H. Pegram, President, Am. Soc. C. E., and provided a simple and efficient method of obtaining the necessary platform facilities for the express service. Before the reconstruction, such a station had two side platforms, one for up-town and one for down-town local traffic. In order to provide access to and from platforms for the express track, the grade of the center track was raised above that of the local tracks so that platforms to serve this track could be built above the local tracks. The standard car clearance required is 14 ft. 6 in. from the base of rail; the construction height of the express platform deck was made as shallow as possible, in order to obtain the least possible distance for the passengers to ascend. The necessary construction height was about 1 ft., and, as the standard height of the platform above the base of rail is 3 ft. 11 in., the grade of the express track was constructed about 11 ft. 6 in. above the local tracks. The grade of the express track was kept parallel to that of the local tracks, and the approaches were constructed with a maximum grade of about 3% and with vertical curves changing the grade 1% in 100 ft., corresponding to a curve of 10 000 ft. radius. Two stairways generally connect each platform with the corresponding local platforms. No additional station building rooms were required, as the old buildings contained all necessary facilities.

Work Divided into Sections.

For executive purposes, the work under "Manhattan Elevated Improvements" was divided into a number of sections as shown on Fig. 2.


SECTION MAP
Express Stations
BROOKLYN
QUEENS
ASTORIA
CORONA
BRONX
NEW JERSEY
HUDSON RIVER
EAST RIVER
HARLEM RIVER
Central Park
Van Cortlandt Park
Woodlawn Cemetery
Bronx Park
CITY LINE
Williamsburg Bridge
Manhattan Bridge
Brooklyn Bridge
Queensboro Bridge
HUNTERS POINT AVE.
DAVIS AVE.
DITMARS AVE.
SYCAMORE AVE.
Cortlandt St.
Warren St.
Desbrosses St.
Christopher St.
Fordham Road
177th St.
149th St.
145th St.
106th St.
116TH
125TH
133d St.
138th St.
143d St.
86th St.
66th St.
59TH
53D
42D
34TH
23D
14TH
SECOND AVE.
THIRD AVE.
SIXTH
NINTH
WEST B'WAY
GREENWICH
SEDGWICK AVE.
JEROME AVE.
WESTCHESTER AVE.
WHITE PLAINS RD.
GUNHILL
E. 162d St.
E. 180TH ST.
E. 194TH ST.
FIG. 2.

As the type of structure and the class and condition of the work varied greatly, the description, for the sake of clearness, will be made for each section separately. The location of each section and the work involved is as follows:

Section No. 1.—This comprised the reconstruction of the Third Avenue Line on Park Row from the terminus at City Hall to Chatham Square, a total length of 1 960 ft., and on New Bowery from near Franklin Square to near Division Street, a total length of 1 840 ft.

The reconstruction in Park Row, including the City Hall Station, was practically a replacement of the old two-track structure with a new four-track structure, only some track stringers and platform girders of the old structure being retained. The old station, just north of Chatham Square, which served all Third Avenue trains, was removed, and a new station with two island platforms and four tracks was built in Chatham Square south of the old location to serve Third and Second Avenue trains to City Hall.

The work in New Bowery included the addition of a two-track structure, above the level of the present structure, supported on new cross-girders carried by the existing columns, and the reconstruction of the South Ferry Branch Station at Chatham Square to provide a train platform for the new Third Avenue Line upper-grade tracks, while the lower-level tracks served the Second Avenue Line.

The work also included the replacing of 52 existing column foundations with larger and deeper ones to carry the additional weight.

Section No. 2-A.—This comprised the reconstruction of the Third Avenue Line in the Bowery from Chatham Square to a point north of Delancey Street, a total length of 2 531 ft., and involved the replacement of the existing structure with a new five-track structure from Chatham Square to Canal Street and a three-track structure from Canal Street to Delancey Street. It included the building, at Canal and Grand Streets, of new express stations to take the place of old ones. Practically all the old structure was removed, the principal exception being the retention of a few of the old track stringers south of Canal Street. In addition to this work, 17 old column foundations on this section were replaced with new foundations.

Section No. 2-B.—This comprised the complete replacement of the old Third Avenue elevated structure in the Bowery from a point north of Delancey Street to Fifth Street, a total length of 2 728 ft. The old

structure consisted of two single-track lines, each supported on a single row of columns at the curb lines of the sidewalks. The new structure, which carries three tracks, is supported on a double row of columns in the roadway. The work in this section also involved the construction of a new express station to take the place of the old one at Houston Street.

Section No. 3.—This comprised the work on the Third Avenue Line in Third Avenue between 5th and 116th Streets. It included the construction of 5 104 lin. ft. of track structure to complete the center track, a portion of which was in existence prior to the commencement of this work, and involved placing new track stringers, replacing the existing cross-braces with new cross-girders, and reinforcing the column tops. The work also included the construction of new over-grade express stations at 9th, 23d, 42d, and 106th Streets, involving the replacement of 35 cross-girders and 70 columns under the stations with new and heavier material.

Section No. 4-A.—This comprised the completion of the center track on the Second Avenue Line from Chatham Square to 116th Street. Portions of the center track were built before the beginning of this work. The new work comprised the construction of 23 382 lin. ft. of new center-track structure, consisting of longitudinal track girders and bracing, and rebuilding the stations at 14th, 42d, and 86th Streets, to provide platforms for the express service, and the reconstruction of the 92d Street Station, which had a center platform, to provide room for the center track.

In this work is not included the reconstruction work between 50th and 60th Streets to connect to the Queensboro Bridge, as that does not form part of the "Manhattan Elevated Improvements."

Section No. 5-A.—This comprised the work to be done on the Third Avenue Line from 116th to 129th Street, and in 129th Street between Third and Second Avenues. The work involved the construction of an over-grade track commencing near 121st Street, connecting with the upper-grade tracks of the new Harlem River Bridge near Second Avenue, and included the replacement of 24 cross-girders and 48 columns with new and heavier structures, the reconstruction of the stations at 125th and 129th Streets, and the replacement of 26 column foundations.

Section No. 5-B.—This comprised the work to be done on the Second Avenue Line in Second Avenue between 116th and 129th

Streets, and included raising the whole structure for a length of 1 387 ft., the maximum rise being 7.5 ft.; the construction of a new express station at 125th Street to replace the existing station at 127th Street, which was removed, and changing the three-track structure north of 126th Street to a four-track structure, with two of the tracks connecting with the upper deck of the new Harlem River Bridge and the other two tracks connecting with the Company's Yard at 128th Street and the lower deck of the new Harlem River Bridge.

Section No. 5-C.—This comprised the replacement of the Harlem River Bridge at 129th Street and Second Avenue, consisting of two approach spans, each about 103 ft. long, and a center swing span, about 248 ft. long, carrying two tracks, with a new double-deck structure, carrying two tracks on each deck.

Section No. 5-D.—This comprised the addition of two upper-grade tracks to the structure from the north end of the Harlem River Bridge through the New York, New Haven and Hartford Railroad Company's Yard to 132d Street, and through the Company's Yard and property between 132d and 133d Streets. The new tracks were supported on new columns, which, in a number of cases, necessitated cutting the existing cross-girders of the lower-deck structure.

Section No. 6-A.—This comprised the work required on the Third Avenue Line from 133d Street, through the Company's private right of way, to 145th Street and through Third Avenue to 147th Street. The work included shifting the existing south-bound track sidewise for 13 ft. for a length of 2 800 ft.; constructing a new two-track overgrade structure between 133d and 143d Streets; reconstructing the stations at 133d, 138th, and 143d Streets, and constructing an overgrade center track between 143d and 147th Streets; and involved the building of new piers and foundations.

Section No. 6-D.—This comprised the work on the Third Avenue Line between 147th Street and Fordham Road, involving the completion of the center track structure; replacing ten island-platform stations with new side-platform stations; reconstructing three stations as express stations; and replacing all the columns supporting the structure between 147th and 177th Streets with new and heavier columns.

Section No. 7.—This comprised the work on the Ninth Avenue Line required to reconstruct the stations at 66th, 116th, 125th, and 145th Streets to provide a continuous center track and express platforms.

Section No. 8-A.—This comprised the work on the Ninth Avenue Line required to construct a center track in Greenwich Street from Cortlandt to 9th Street, and center platforms at Cortlandt, Warren, Desbrosses, and Christopher Streets, and involved the construction of 10 540 lin. ft. of new track structure, consisting of track stringers and cross-girders; reinforcement of column tops; the replacement of 154 columns, and of 4 328 lin. ft. of old longitudinal girders with new girders for the existing south-bound track.

Section No. 8-B.—This comprised the reconstruction of the existing stations on the Ninth Avenue Line at 14th and 34th Streets to provide platforms for the express track.

Section No. 8-C.—This comprised the construction of an over-grade crossing of the Ninth Avenue express track over the junction with the Sixth Avenue tracks at 53d Street and Ninth Avenue.

Section No. 10-B.—This comprised the reconstruction of the terminal station of the Ninth Avenue Line at 155th Street and Eighth Avenue, and involved the addition of an over-grade track between 150th and 155th Streets, the construction of two island platforms between 155th and 157th Streets, and the reconstruction of the track arrangement between 154th Street and the Company's Yard at 159th Street. A portion of this work cannot be completed until the structure on the east side of Eighth Avenue north of 155th Street, which is occupied by the Putnam Division of the New York Central Railroad, is vacated.

Design of Steel Structure.

The purpose of the work under the "Manhattan Elevated Improvements" was to improve the existing traffic facilities. The new track arrangement, therefore, was a matter of first consideration. When that was settled, the existing structure was surveyed for the purpose of determining its strength; the reinforcement needed, where new loads were added; the dimensions of the added structure, and the methods of connections between new and old work. The necessity of maintaining traffic during reconstruction was the governing feature in the design.

Specifications for Design.

The specifications for loads and unit stresses for the additions to the structure north of the Harlem River were different from those for the added structure in Manhattan, because the original structures

were built under different specifications, and it was desired to keep the stresses in the old and new structure uniform. The specifications for the new structure added to the elevated lines in Manhattan were as follows:

Dead Load.—The dead load shall consist of the estimated weight of the entire suspended structure, and is estimated to be 750 lb. per lin. ft. of track.

Live Load.—The live load for each track shall consist of a train of cars with 6-ft. wheel base, 33 ft. from center to center of trucks, and 15 ft. from center to center of adjacent cars, the cars having 20 000 lb. on each axle of the north truck and 15 000 lb. on each axle of the south truck.

The live load for canopy roofs shall be 30 lb. per sq. ft., and for station platforms, 80 lb. per sq. ft.

Lateral Load.—Transverse bents shall be designed for a force of 30 lb. per sq. ft. on the exposed surface of all trusses and the floor, as seen in elevation, and on the side of a train 10 ft. high, beginning 3 ft. above the base of rail.

Longitudinal Force.—Transverse bents and similar structures shall be designed for a longitudinal force of 10% of the live load applied to the rail.

Structures on curves shall be designed for the centrifugal force of the live load acting at a height of 5 ft. above the rail, proper account being taken of the super-elevation, to be determined by the Engineer for each case.

The centrifugal force is $f = \dfrac{W v^2}{g r}$,

in which $f =$ pounds,
$v =$ velocity, in feet per second,
$W =$ weight, in pounds, and
$r =$ radius, in feet.

Unit Stresses.—All structures designed for additional tracks or stations on existing elevated railroad structures in Manhattan shall be proportioned so that the maximum stress in axial tension on the net section shall not exceed 9 000 lb. per sq. in.

The axial compression on the gross section is $9\,000 - 40 \dfrac{l}{r}$,

in which l is the length of the member, and r is the least radius of gyration, both in inches.

No column smaller than 12 in. shall be used.

Bending.—On extreme fibers of rolled shapes, built sections, and girders, net section, 9 000 lb. Roof work shall be designed for a fiber stress of 12 000 lb. per sq. in.

Shearing.—Rivets	....	7 500 lb.
Plate girder webs (gross section)	...	7 500 "
Bearing	....	15 000 "

For stiffeners, ground to bear, deduct area lost by grinding for fillet, in calculating effective area of stiffeners.

Bearing on Base Plates.—For direct loads, base plates of columns shall be proportioned for a pressure of 300 lb. per sq. in. on concrete (1:2½:5), adding 50% to this pressure for combined direct and bending stresses.

Bending.—On extreme fibers of rivets and pins, 15 000 lb.

Members subject to alternate strains of tension and compression shall be proportioned for the strain giving the largest section. If the alternate strains occur in succession during the passage of one train, as in stiff counters, each strain shall be increased by 50% of the smaller.

Whenever the live and dead-load strains are of opposite character, only 70% of the dead-load strain shall be considered as effective in counteracting the live-load strain.

Members subject to both axial and bending strains may be proportioned to a combined fiber strain 25% in excess of the allowed axial strain.

For strains produced by longitudinal and lateral or wind forces, combined with those from live and dead load and centrifugal forces, the unit strain may be increased 50% over the allowed axial strain. The section, however, shall not be less than that required if the longitudinal and lateral or wind forces are neglected.

Plate girders shall be proportioned either by the moment of inertia of their net section, or by assuming that the flanges are concentrated at their centers of gravity, in which case one-eighth of the gross section of the web, if properly spliced, may be used as the flange section.

The gross section of the compression flange of the plate girders shall not be less than the gross section of the tension flanges, nor shall

the strain per square inch in the compression flange of any beam or girder exceed $12\,000 - 200\frac{L}{B}$, in which $L =$ the unsupported distance, and $B =$ the width of the flange.

The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly to the flange. The wheel load, where the ties rest on the flanges, will be assumed to be distributed over three ties.

Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams, used as girders, shall preferably have a depth of not less than one-twelfth of the span, and if shallower trusses, girders, or beams are used, the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratio had not been exceeded.

The specifications for the structure north of the Harlem River varied from the foregoing as follows:

Dead Load.—The dead load shall consist of the estimated weight of the entire suspended structure, and is estimated to be 750 lb. per lin. ft. of track.

Live Load.—The live load for each track shall consist of a train of cars, the cars being 46 ft. long, 32 ft. from center to center of trucks, 14 ft. from center to center of trucks of adjacent cars, 5 ft. from center to center of wheels of the same truck, having 25 000 lb. on each of the four axles.

The live load for canopy roofs shall be 30 lb. per sq. ft., and for station platforms, 80 lb. per sq. ft.

Unit Stresses.—All new structures shall be proportioned so that the maximum stress in axial tension on the net section shall not exceed:

In longitudinal girders, 10 000 lb. per sq. in.
In cross-girders, 12 500 " " " "

Axial compressions in columns.............. $10\,000-40\,\frac{l}{r}$

" " " longitudinal girders ..$11\,000-40\,\frac{l}{r}$

" " " cross-girders.........$12\,500-40\,\frac{l}{r}$

in which l is the length of the member, and r is the least radius of gyration, both in inches.

No column smaller than 12 in. shall be used.

Bending.—On extreme fibers of rolled shapes, built sections, and girders, net section:

Longitudinal girders .... 11 000 lb.

Cross-girders .... 12 500 lb.

Standards for Designs.—The standard overhead clearance is 14 ft. 0 in. above the top of rail, and the side clearance, 6 ft. 0 in. from center of track, except at platforms, where it is 4 ft. 7½ in.

The standard Manhattan car is 12 ft. 10½ in. high and 8 ft. 9½ in. wide. The track gauge is 4 ft. 8½ in.

Bending Moments in Statically Indeterminate Bents.—All cross-bents, except in special cases, where tower construction could be obtained were generally designed to resist moments both at the top and bottom of columns. In the course of the design, one, two and three-story bents were encountered, and for each type formulas for the moments at the top and bottom of columns were worked out. As the results may be of general interest, they are noted here for each type considered.

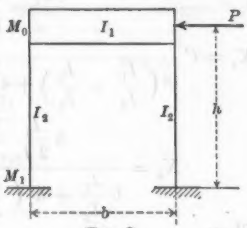


FIG. 3.

Type I.—Single-Story Bent; Both Columns Alike.

Let h = height of column, measured from bottom of column to center of cross-girder;

b = distance between centers of columns;

I_1 = moment of inertia of cross-girder;

I_2 = " " " " column;

P = horizontal force applied at center of cross-girder;

M_0 = moment at top of column;

M_1 = moment at bottom of column.

Then

$$M_0 = -\frac{3\,I_1\,P\,h^2}{2\,(b\,I_2 + 6\,I_1\,h)}$$

and

$$M_1 = +\frac{b\,I_2 + 3\,h\,I_1}{2\,(b\,I_2 + 6\,I_1\,h)}\,P\,h.$$

Type II.—Unsymmetrical One-Story Bent.

Let h = height of column, measured from bottom of column to center of cross-girder;

b = distance between centers of columns;

I_1 = moment of inertia of one column;

I_2 = " " " " other column;

I_3 = " " " " cross-girder;

M_0 = moment at bottom of column I_1;

M_1 = " " top " " I_1;

M_2 = " " " " " I_2;

M_3 = " " bottom " " I_2;

P = horizontal force applied at center of cross-girder;

and let

$$X_1 = P \frac{6\,h^2\left(b + h\left(\frac{I_3}{I_1} + \frac{I_3}{I_2}\right)\right)}{b^2\left(\frac{I_1}{I_3} + \frac{I_2}{I_3}\right) + 4\,b\,h\left(\frac{I_1}{I_2} + \frac{I_2}{I_1} + 2\right) + 12\,h^2\left(\frac{I_3}{I_1} + \frac{I_3}{I_2}\right)b}$$

$$X_2 = \frac{b\,\frac{I_1}{2\,I_3} + h}{b\,\frac{I_1}{I_3} + h\left(\frac{I_1}{I_2} + 1\right)}$$

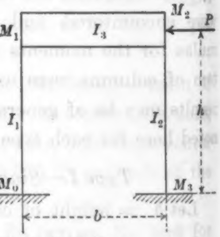


FIG. 4.

Then

$$M_0 = -X_1\,(b - X_2) + \frac{P\,I_1\,h}{I_1 + I_2}$$

$$M_1 = -X_1\,(b - X_2)$$

$$M_2 = X_1\,X_2$$

$$M_3 = X_1\,X_2 - \frac{P\,I_2\,h}{I_1 + I_2}$$

Type III.—Two-Story Bent; Both Stories Same Width; All Columns Alike.

Let h_1 = distance from center to center of cross-girders;

h_2 = distance from center of lower girder to bottom of column;

b = distance between centers of columns;

I_1 = moment of inertia of upper cross-girders;

I_2 = " " " " " column;

I_3 = " " " " " lower cross-girders,

I_4 = " " " " " column;

M_a = moment at top of upper column;

M_b = moment at bottom of upper column;
M_c = " " top " lower "
M_d = " " bottom " " "
P_1 = horizontal force applied at center of upper cross-girder;
P_2 = " " " " " " lower "

Further, let

$$A = \frac{P_1 h_1^2}{4 I_2}$$

$$B = \frac{P_1 h_2^2 + 2 P_1 h_1 h_2 + P_2 h_2^2}{4 I_4}$$

$$C = \frac{b^2}{12 I_1} + \frac{b h_1}{2 I_2} + \frac{b h_2}{2 I_4}$$

$$D = \frac{b h_2}{2 I_4}$$

$$E = \frac{b^2}{12 I_3} + \frac{b h_2}{2 I_4}$$

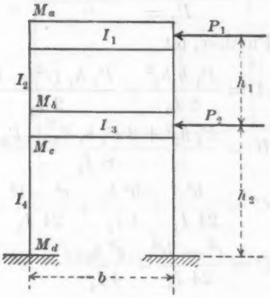


FIG. 5.

and

$$X_1 = \frac{(A + B) E - B D}{C E - D^2}$$

$$X_2 = \frac{B C - (A + B) D}{C E - D^2}$$

Then

$$M_a = -\frac{b}{2} X_1$$

$$M_b = -P_1 h_1 - \frac{b}{2} X_1$$

$$M_c = +P_1 h_1 - \frac{b}{2} (X_1 + X_2)$$

$$M_d = +P_1 (h_1 + h_2) + P_2 h_2 - \frac{b}{2} (X_1 + X_2)$$

Type IV.—Two-Story Bent; Upper Story Narrower than Lower Story; Symmetrical.

Let h_1 = distance from center to center of cross-girders;
h_2 = " " " of lower cross-girder to bottom of column;
b = distance between centers of columns of upper bent;
c = " " " " " lower "
d = " from center of column of upper bent to center of column of lower bent;

I_1 = moment of inertia of upper cross-girder;
I_2 = " " " " " column;
I_3 = " " " " lower cross-girder;
I_4 = " " " " " column;
P_1 = horizontal force applied at center of upper cross-girder;
P_2 = " " " " " " lower "

Further, let

$$A = \frac{P_1\, b\, h_1^2}{8\, I_2} + \frac{P_1\, h_1\, (c^2 - b^2)}{2\, I_3}$$

$$B = \frac{P_1\, h_2^2 + 2\, P_1\, h_1\, h_2 + P_2\, h_2^2}{8\, I_4}\, c$$

$$C = \frac{b^3}{24\, I_1} + \frac{b^2\, h_1}{4\, I_2} + \frac{c^3 - b^3}{24\, I_3} + \frac{c^2\, h_2}{4\, I_4}$$

$$D = \frac{c^3 - b^3}{24\, I_3} + \frac{c^2\, h_2}{4\, I_4}$$

$$E = \frac{c^3}{24\, I_3} + \frac{c^2\, h_2}{4\, I_4}$$

$$F = \frac{P_1\, h_1\, (c^2 - b^2)}{16\, I_3}$$

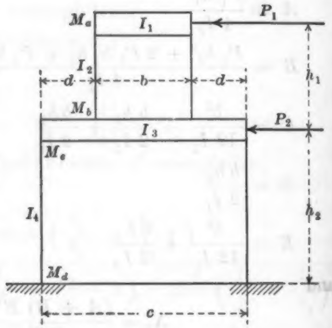


FIG. 6.

and let

$$X_1 = \frac{(A + B)\, E - (F + B)\, D}{C\, E - D^2}$$

$$X_2 = \frac{(F + B)\, C - (A + B)\, D}{C\, E - D^2}$$

Then

$$M_a = -\frac{b}{2} X_1$$

$$M_b = -\frac{b}{2} X_1 + \frac{P_1\, h_1}{2}$$

$$M_c = -\frac{c}{2} (X_1 + X_2) + \frac{P_1\, h_1}{2}$$

$$M_d = -\frac{c}{2} (X_1 + X_2) + \frac{P_1 (h_1 + h_2)}{2} + \frac{P_2\, h_2}{2}$$

Type V.—Three-Story Pin-Connected Bent.

Let h_1 = height of upper story;
h_2 = " " middle "
h_3 = " " lower "
b = distance between centers of columns;

M_1 = moment at bottom of upper column;

M_2 = moment at bottom of middle column;

M_3 = moment at bottom of lower column;

P = horizontal force applied at top of bent;

and let

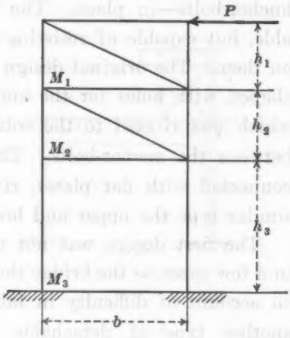


FIG. 7.

$$p = \frac{h_2 h_3 (4 h_1 + 3 h_2) + 6 h_3^2 (h_1 + h_2)}{h_2 (4 h_1 + 3 h_4) + 12 h_3 (h_1 + h_2)}$$

$$q = h_3 + \frac{2 h_2 (h_1 + h_2)}{2 h_1 + 3 h_2}$$

$$X = P \left(1 - \frac{3 h_2 h_3^2}{h_1 (4 h_1 h_2 + 3 h_4^2 + 12 h_1 h_3 + 12 h_4 h_3)}\right)$$

Then

$$M_1 = + \frac{1}{2} (P - X) h_1$$

$$M_2 = - \frac{1}{2} P (h_3 - p)$$

$$M_3 = - \frac{1}{2} Pp$$

GENERAL DETAILS OF DESIGN.

New columns, supporting the structure in the street, were generally made 15 in. square, built up of two 15-in. channels with the flanges turned in, four angles, and a web-plate. Using 50-lb. channels, 5 by 3½ by ¾-in angles, and 13 by ½-in. web, the section modulus of such a column is 129 in.³ in the direction parallel to the web and 294 in.³ in the direction square to the web. The columns were nearly always placed with the web in the same direction as the cross-girders, as the lateral bending stresses in the columns, due to wind and centrifugal forces, are greater than the longitudinal bending stresses, due to braking of trains. Columns on the sidewalk, supporting station structure only, were generally built up similar to the track structure columns, but 12 in. square.

Generally, the columns had to be designed so that they could be erected under the cross-girder, with the foundations—including the

anchor-bolts—in place. The columns, therefore, were made detachable, but capable of resisting the bending moments that might come on them. The original design had a lower part made up of plates and shapes, with holes for the anchor-bolts. The upper part of the base, which was riveted to the column, was made narrow enough to pass between the anchor-bolts. The upper and lower sections were to be connected with flat plates, riveted in the field. In the design of a similar type the upper and lower parts are connected with bolts.

The first design was not used at all, and the second design only in a few cases, as the bridge shops requested that the design be changed, on account of difficulty in fabricating the lower sections. Therefore, another type of detachable base was designed. The lower part of this base consisted of a rolled steel slab, 6 in. thick, with four holes to fit over the anchor-bolts in the foundation. The upper part of the base, which was narrow enough to pass between the anchor-bolts, was riveted to the column shaft, and was connected to the slab with four screw-bolts, 2 in. in diameter, fitting into tapped holes in the slab.

A stiff connection between the column and the cross-girder was obtained, when the column was at the end of the cross-girder, by running the column shaft to the top of the cross-girder and riveting the end of the girder to the column. When the column was under the cross-girder, the stiff connection was obtained by riveting four angles to the column, extending above the top of the column; these angles were connected to the bearing stiffeners of the cross-girder with plates riveted to both angles and stiffeners. The column and the cross-girder were further connected with plate brackets riveted to the sides of the column and the bottom of the cross-girder.

The cross-girders were generally plate girders in preference to lattice girders, due to the simplicity of design and also the facility with which a load can be placed at any point on a plate girder, without disturbing the uniformity of the design. This is specially advantageous for a structure of the kind herein described, as the location of the loads, due to track stringers, platform girders, overhead structures, etc., may vary from bent to bent.

The track stringers also were usually plate girders, but, where it was desired, either to keep the construction as open as possible, or to conform to the existing design, lattice stringers were used. Whenever

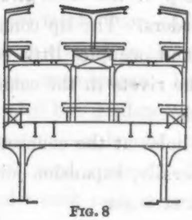


FIG. 8

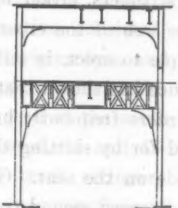


FIG. 9

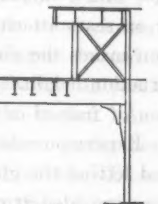


FIG. 10

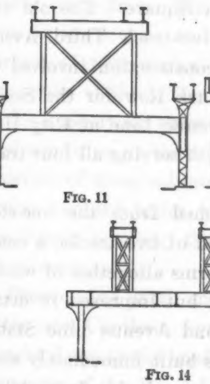


FIG. 11

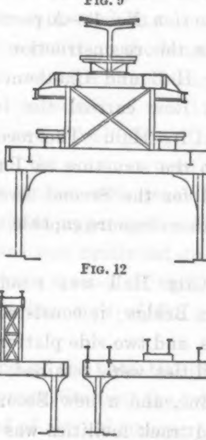


FIG. 12

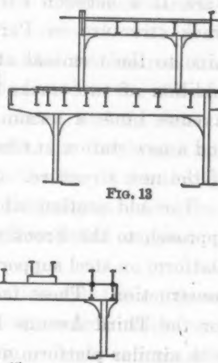


FIG. 13

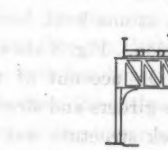


FIG. 14

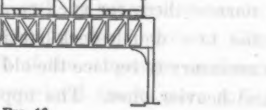


FIG. 15

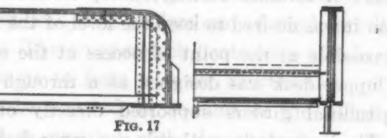


FIG. 16

FIG. 17

the conditions permitted, the track stringers were designed with end lips, which supported the stringers, either on top of the cross-girders or on seats attached to the web of the cross-girders. The lip connection makes the girder simple to erect, is stiff, and occupies little construction height. Experience has shown that the rivets in the connection of framed-in track girders frequently break.

Expansion was provided for by slotting the holes at the connection and letting the girders slide on the seat. Generally, expansion joints were provided at one end of every second track stringer.

Details of Design.—Section No. 1.—A portion of the work included under Section No. 1 was the reconstruction of the elevated line in Park Row between City Hall and Chatham Square. The old two-track structure in Park Row carried the two-track Third Avenue Line to the terminal at City Hall. The reconstruction involved the addition of two tracks to the structure in Park Row for the Second Avenue Line; a terminal for the Second Avenue Line at City Hall; and a new station at Chatham Square capable of serving all four tracks of the new structure.

The old station at City Hall was reached from the one-story approach to the Brooklyn Bridge; it consisted of two tracks, a center platform on steel supports, and two side platforms altogether of wooden construction. These facilities were retained, but improved in details for the Third Avenue Line, and a new Second Avenue Line Station with similar platform and track facilities was built immediately above the Third Avenue Line Station, thus making a double-deck station. There was no room to carry all four tracks at one level, because Park Row is too narrow there for such construction. Fig. 8 shows a cross-section of the new double-deck station. On account of the added load, it was necessary to replace the old cross-girders and street columns with new and heavier ones. The upper-deck structure was supported on four rows of columns resting on top of the new lower-deck cross-girders. As it was desired to keep the level of the upper-deck platforms as low as possible at the point of access at the south end of the stations, the upper deck was designed as a through bridge construction, with longitudinal girders supported directly on the four rows of columns. The lower deck was level; the upper deck was laid on a grade of ½% rising from the south to the north end of the station. This was done in order to gain sufficient clearance at the north end for

the standard cross-girder construction where it was required, on account of the cross-overs from in-bound to out-bound tracks. The upper-track floor construction consisted of shallow cross-girders or beams riveted to the through girders, and track stringers framed into these cross-beams. In order to make the floor as shallow as possible, riveted box girders were used at the south end of the station. As the construction height increased, 15 and 18-in. Bethlehem girder beams were used instead. The track stringers consisted of two I-beams under each rail, in order to get sufficient bearing surface for the ties. The center platform was supported on top of the longitudinal girders, and the outside platforms partly on the longitudinal girders and partly on the track floor. The columns supporting the upper-deck structure were 12 in. square, and consisted of two 12-in. channels with a diaphragm of four angles and a web-plate. In order to obtain a fixed connection at the bottom, the columns were connected to the stiffeners of the supporting cross-girders with tension plates, and to obtain a fixed condition at the top of upper-deck columns, the diaphragm construction of these columns was continued up to the top of the through girders and the latter were riveted to these diaphragms. At expansion joints, however, the free ends of the girders rested on seats on the columns.

Between the north end of the City Hall Station and the south end of the new Chatham Square Station, the two upper-deck tracks were brought down to the level of the old structure, in order to connect with the tracks of the Second Avenue Line at Chatham Square. This was accomplished by running the lower-deck tracks to the west side of the structure through Park Row, and the upper-deck tracks to the east side. The first few bents of the upper deck north of City Hall Station had to span the entire width of the structure, as shown by Fig. 9, but as soon as the upper and lower-level tracks were sufficiently clear of each other, the upper deck was supported on two-column braced bents supported on the lower-deck cross-girders east of the lower-deck tracks, as shown by Fig. 10. These upper-deck bents were also braced longitudinally in pairs, so as to form a tower construction. The lower-deck columns had to be renewed on account of the added load, and where they coincided in position with the upper-deck columns they were built in one piece from the street to the upper deck. For appearance sake, it was desired to avoid transverse bracing below the lower

deck; as, however, the regular column section below the lower deck was not sufficient to carry the combined direct and transverse bending stresses without bracing, and as it was not permitted to use a width of column greater than 15 in. where it might interfere with the use of the sidewalk, the necessary additional strength was obtained by widening the column from a point 12 ft. above the sidewalk. This was accomplished by using two angles on the street side of the columns instead of the customary 15-in. channel. At a distance of 12 ft. above the sidewalk, the web was spliced and its upper part extended through the angles and reinforced with flange angles. Some of the existing lower-deck lattice cross-girders were retained, where there was no additional loading, but, on account of the track girders being changed in position, the latticing of cross-girders was removed at such points, and a web-plate was riveted to the cross-girder instead, as shown by Fig. 9.

The remainder of the work under Section No. 1 comprised the construction of an upper-grade station above the existing station on the east side of Chatham Square and a two-track approach to this station both at the north and south ends. The old lower-deck station was formerly used for both Second and Third Avenue Lines on the South Ferry Branch. After the reconstruction the lower deck was used for Second Avenue Line trains only, and the new upper deck for Third Avenue Line trains. The approach to the upper station commenced in the New Bowery just north of Franklin Square Station. The structure involved in the New Bowery consisted, before the reconstruction, of two single rows of columns on the sidewalks, each row carrying a single-track structure. The old columns opposite were braced with stiffening struts across the street. The columns were of the "fan-top" type, that is to say, they were made of two channels latticed together, and the channels were flared at the top, so that a track stringer came directly on top of each channel.

The new tracks were carried on cross-girders, which, at the beginning of the ramp at the south end, were set on the top of the old columns between the old track stringers, because the head-room above the street was small. These cross-girders were made in three pieces, in order to facilitate the erection. Farther north, where the headroom was greater, the new cross-girders were framed into the sides of the column shaft, as shown by Fig. 11; this method was preferred,

because the new structure could be erected without shoring and without interfering with traffic. In order to transfer the additional loading uniformly to the column, a diaphragm was riveted inside of the channels of the column, thus carrying the cross-girder reaction equally into both sides of the column. The upper tracks were supported by columns centered on the tracks and resting on the new cross-girders. The two columns of each bent were braced together, and every second pair of bents was also braced together, so that the upper structure formed a completely braced tower construction, resting on top of the new cross-girders. Additional longitudinal bracing was also provided between the old columns, so as to decrease the unsupported length of these columns and thus add to their carrying capacity.

Fig. 12 shows a cross-section of the structure at the Chatham Square Station. On account of the additional loading, it was necessary to replace the existing columns and cross-girders with new and heavier ones. This figure shows a cross-section at the south end of the station, where the lower center platform is divided into two platforms with an open space between. At the crossing with the City Hall branch tracks, the track layout necessitated a span length of about 75 ft. A through-bridge construction, as shown by Fig. 13, was used in order to obtain sufficient depth for the stringers without encroaching on the head-room. The location of the columns was determined by the track arrangement on the lower deck.

Details of Design.—Section No. 2-A.—The work under Section No. 2-A comprised the reconstruction of the Third Avenue Line in the Bowery between Chatham Square and a point north of Delancey Street. Prior to the reconstruction, the old Chatham Square Station of the Third Avenue Line was in the Bowery just north of Chatham Square. The old station had a wide island platform serving the existing tracks. The northerly part of the platform was divided through the center, and the space was occupied by two pocket tracks, with an island platform between them, raised slightly above the grade of the main structure. The pocket tracks ended just south of the old Canal Street Station, where they turned into the main tracks. North of this point, the line was a two-track structure. The old Canal Street Station had two side platforms extending to the south of Canal Street. North of Canal Street, the structure consisted of two rows of fan-top columns support-

ing the track structure. The old Grand Street Station was between the tracks, with the platforms extending to the south of Grand Street.

The new track layout south of Canal Street retained the old main tracks in approximately their old location for local service on the City Hall branch. A center track was added on the same level for express service on the City Hall branch, and between the center track and the local tracks were added two over-grade tracks to connect with the South Ferry branch. The old pocket tracks and the station north of Chatham Square were removed. Fig. 14 shows a cross-section of the new structure just north of Chatham Square. On account of the additional loading, the existing columns and cross-girders had to be replaced with new and heavier ones. The existing track level, which it was necessary to maintain at the Chatham Square Junction, was close to the street, so that the new cross-girders had to be made very shallow compared with their span, in order to obtain the required 14 ft. of head-room under the structure. The use of twin girders was first considered, but as a plate-girder construction was practically necessary, this type of girder was abandoned on account of the difficulties in making connections to and maintaining a box-girder construction. Therefore, single plate girders, with heavy flanges, 20 in. wide, were used. On account of the width of the flanges, it was necessary to place track ties on top of the cross-girder, in order not to space the ties too far apart. The tops of the cross-girders, therefore, were made flush with the tops of the track stringers, and the latter were supported on seats attached to the web of the cross-girders and had the same depth.

The upper-level tracks were supported separately by framed two-post bents of the same width as the distance between the supported track stringers. The bents on two adjacent cross-girders were braced together longitudinally, and, in order to prevent sidewise overturning, the bents were spliced to the supporting cross-girders. The upper local track stringers were supported on the bents with lips 18 in. deep.

In order to facilitate the construction, the old locations of the columns on the sidewalk were retained; this, however, involved skew connections between the columns and the cross-girders, as well as between the cross-girders and the track stringers.

At the south end of the section, where the existing track grade was maintained, the old track stringers of the existing main tracks were used in the new work, after the stringer ends had been remodeled to

match the new cross-girders. Farther north, where the new track grade was higher than the old one, new track stringers were used, because the maintenance of traffic necessitated, first, the erection of the new cross-girders and the framing of the track stringers at the existing level, and, later, the raising of the track stringers to the new level. To avoid this double reconstruction of the old track stringers, which would have been unsatisfactory and expensive, they were remodeled for the temporary connection to the new cross-girders, and new stringers were provided for the final position. Fig. 15 shows the temporary and permanent arrangement at the connection of the stringers and the cross-girder. The old stringers were carried temporarily on seats supported by the column brackets. When the old stringers were removed, additional stiffeners were riveted to the cross-girder at points where the new stringers were supported on the top of the cross-girder. Fig. 15 also shows the seats supporting the upper deck, at a point where the upper grade approaches the elevation of the lower deck.

The new express station at Canal Street was built as a mezzanine station. In order to obtain sufficient head-room for the mezzanine structure, the new track grade was raised above the old grade. After the design was completed, the City authorities decided to raise the grade of Canal Street in conjunction with the approach to the Manhattan Bridge. This necessitated a further raising of the new track level, which was obtained by a shortening of the vertical curves.

Fig. 16 shows the cross-bents supporting the mezzanine station. Whenever it was possible, the columns supporting the structure were placed in the roadway. The cross-girders were 8 ft. deep, and the mezzanine station occupied the full depth of the girders, the bottom of the mezzanine floor being nearly flush with the bottom of the girders. The longitudinal girders in this span were set on top of the cross-girders and carried the mezzanine station, the platforms, and a through bridge construction for supporting the track. Fig. 17 shows the details of the longitudinal girders. They were plate girders, and the web was extended below the bottom flange of the girder in order to provide means for connecting the hangers supporting the mezzanine structure.

A cross-section of the remainder of the structure of the Canal Street Station is shown by Fig. 18. The track stringers are carried on seats attached to the web of the cross-girder, and the top of the stringers is 3 in. below the back of the top flange angles of the cross-girders. This

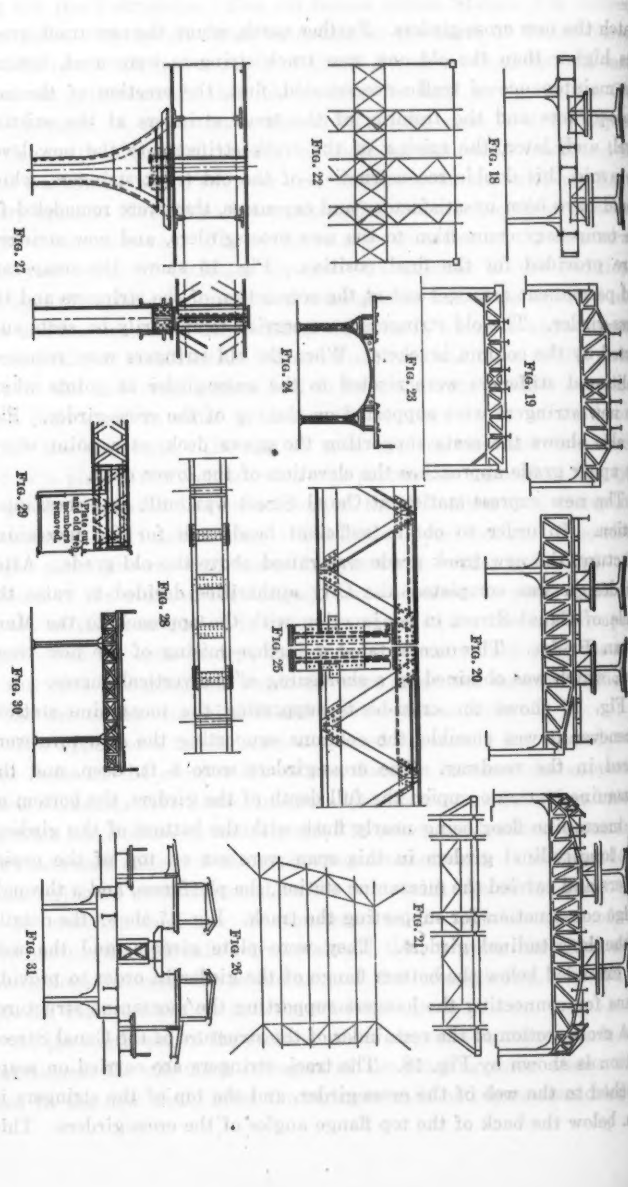


FIG. 18

FIG. 19

FIG. 20

FIG. 21

FIG. 22

FIG. 23

FIG. 24

FIG. 25

FIG. 26

FIG. 27

FIG. 28

FIG. 29

FIG. 30

FIG. 31

arrangement permits the rail to be carried clear of the cross-girder without undue spacing of the ties. The bottom flange of the stringers was made flush with the bottom of the cross-girders, and the bottom flanges of the stringers and cross-girder were connected with a plate. The platform stringers were 4 ft. deep, and rested, with a lip 2 ft. 4 in. deep, on the top of the cross-girders.

The structure was braced with diagonal struts from the center track stringers to the columns, in order to prevent horizontal deflection of the cross-girders due to braking.

From north of the Canal Street Station to the south end of the Grand Street Station, the distance between street-car tracks and curb necessitated placing the columns on the sidewalk. The existing structure consisted of two rows of columns supporting the track structure, one on each side of the street. The new grade of the tracks was considerably higher than the old one on account of the changes of the street grade and the mezzanine stations, but it was necessary to keep the ends of the cross-girders below the existing track work, as traffic on the existing tracks had to be maintained during the erection of the new structure. The cross-girders, therefore, were designed as shown by Fig. 19. The bottom flange of the cross-girders was kept straight; the middle portion of the top flange was kept high, partly to get sufficient depth of the girder and partly to bring the track stringers, which were set on top of the cross-girder, to the proper level. At the ends, the top flange of the cross-girder was brought down to such a level that the girder could be erected without interfering with the ties of the existing tracks.

The construction through the Grand Street Station was similar to that just described except that twin girder construction was used to carry the additional load. Fig. 20 shows one of the bents carrying the mezzanine station at Grand Street. As one of the columns could be placed in the street, it was not necessary to drop the one end of the girder, because the cantilever end was not erected until the traffic on the existing track was abandoned. The seat at the other end of the cross-girder was added after traffic had stopped on the existing track, and was required for the support of the through bridge girder carrying the track floor. The construction of the mezzanine station was the same as that at Canal Street. Fig. 21 shows the details of the bents carrying the platforms outside of the mezzanine station. The seats were made in two pieces, because, on account of the necessity

of maintaining traffic, the platform could not be constructed to its full width at once. In order to prevent longitudinal deflection of the cross-girders, horizontal trusses were provided; they were riveted to the bottom flanges of the stringers, and braced to the columns, as shown by Fig. 21.

From north of Grand Street Station to a point near Delancey Street, bents of the same type were continued, owing to the necessity of keeping the columns on the sidewalks. As the distance from the stringers to the columns was too great to use a diagonal brace, the method of bracing was changed to the type shown by Fig. 22. A horizontal stiffening truss, placed at the center of the span, connected the bottom flanges of all the stringers and extended at each end out to, and was connected with, the top flanges of vertical stiffening trusses between the columns.

Details of Design.—Section No. 2-B.—In this section, which extended from near Delancey Street to near 5th Street, all the columns of the new structure, with the exception of a few near 5th Street, were in the roadway; columns of the old structure were at the curb line, and the whole new structure, therefore, could be erected without interfering with the old one, except at the connection at 5th Street. Fig. 23 is a typical section of the structure. As there were four street-car tracks, the columns of the bent were spaced 45 ft. from center to center. The columns were originally designed with fixed bases, capable of resisting the moments. It developed, however, during the construction, that it would be necessary to let a gas main pass directly through the center of the foundations on both sides of the street, and it was decided, therefore, that it would be safer to place a heavy steel slab on the top of the foundations. The detachable type of base, with steel slab 6 in. thick, therefore, was decided on, and, for the sake of uniformity, this base was used throughout on both sides of the street. The cross-girders were designed as twin lattice girders. The twin girder construction was used because it was desired not to carry the structure higher than necessary. The cross-girders rested with lips on top of the columns, and were braced to the column shaft with angle brackets. The standard spacing of the track stringers was 5 ft. between the stringers of each pair and 12 ft. from center to center of tracks. The panel spacing was arranged so as to bring the panel points directly under the track stringers.

Fig. 25 shows the details of the ends of the track stringers. The girders were supported on the top of the cross-girder with lips. As it was impossible to avoid secondary stresses at the support, the top chord was reinforced with 12-in. channels. Fig. 26 shows the lateral stiffening trusses and the braces to the columns. It will be noted that the braces have the additional function of fixing the tops of the columns, the braces and the stringer to which they are attached acting together as a stiffening truss.

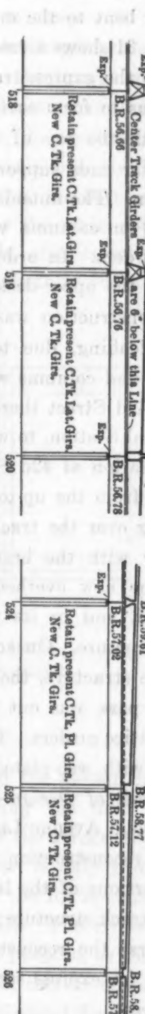
The Houston Street Station is included in this section; its general construction is similar to that of other mezzanine stations on the Bowery. As at the other stations, the depth of the cross-girders was made to conform to the height of the mezzanine. Through bridge girders carried the platform and track construction over the mezzanine station.

Details of Design.—Section No. 3.—Prior to the reconstruction there existed on the Third Avenue Line between 5th and 116th Streets, which portion comprised Section No. 3, a partial center-track construction, in fact, an express service was in operation north of 42d Street. South of the 42d Street Station center-track girders were in existence at a few isolated points, but, generally, the structure consisted of two tracks supported directly on top of the columns and braced together with an arch-shaped strut, as shown by Fig. 24. The columns were of the fan-top type, hereinbefore described. Originally, this structure was designed to carry only the load of the track supported directly by the columns, but, as a center track was added in places, in the course of time, it became necessary to reinforce the flaring portion of the column, and a design for the reinforcement was developed. This design, Fig. 27, was used also for the reinforcement of the columns included in the new work. The general principle of the design was that the load of the stringers supporting the outside tracks should be transferred to the columns as before, and that the load of the center track should be carried by a cross-girder to the center of the column, where it was supported on two I-beams, framed into two channels, riveted to plates, which were riveted to the flaring flanges of the column channels. These plates also strengthened the curved portions of the column channels which took the load of the outside track stringers. This somewhat involved construction was necessary, in order to be able to do the riveting in the closed space between the

channels and the plates. The new cross-girders were plate girders. To facilitate erection, they were made in three pieces, as shown by Fig. 28. The outside track stringers rested on the bottom flange of the cross-girders, and were supported directly by the column. The center track stringers were framed into the cross-girder at the fixed connections, and rested on seats at the expansion end. The new track stringers were plate girders.

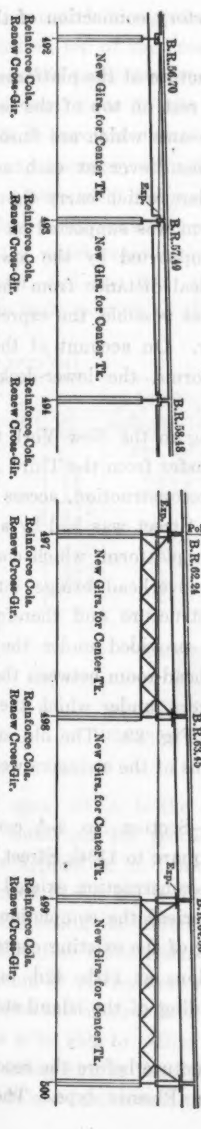
On this section, the stations at 9th, 23d, 42d, and 106th Streets were rebuilt for express service. As there was not sufficient head-room at any of these stations for a mezzanine station, the "hump" type of express stations was used. Fig. 33 shows portions of the southerly approach to the 42d Street Station. As there were no center-track girders at this location prior to the reconstruction, new cross-girders and stringers were provided. The cross-girders had to conform to the elevation of the existing local tracks, while the center-track stringers were rising above the grade of the local tracks. In the first few spans, where the stringers came slightly above the top flange of the cross-girder, they were cut to make room for the cross-girder flange. When raised sufficiently, a lip construction was used until the stringer was clear of the cross-girder. Seats and bents were used thereafter to support the stringers. The bents consisted of two posts, placed 5 ft. from center to center, braced together, and set on top of the cross-girders. The bent was braced longitudinally with two diagonal struts supported on longitudinal lattice stiffening trusses framed into the cross-girders in the fixed spans.

The north approach differs somewhat in certain details, because the cross-girders and center-track stringers were in place prior to this work. The center-track stringers were retained in place, in order to stiffen the structure, and the raised track was constructed on top of these stringers. At the low portion of the approach, tapered girders, as shown by Fig. 32, were riveted to the top flange of the existing stringers. As the height of the new track above the old one became greater, plate-girder seats, resting on top of the track stringers close to the cross-girder, were used, and finally A-shaped towers, with the posts resting on the stringers. The reason for supporting the seats and the towers on the stringer ends, rather than directly on the cross-girder, was that the presence of the old track stringers, framed into



HUMP STRUCTURE NORTH OF 42d STREET, THIRD AVENUE LINE

FIG. 32.



HUMP STRUCTURE SOUTH OF 42d STREET, THIRD AVENUE LINE

FIG. 33.

the cross-girders, did not permit a satisfactory connection of the supporting bent to the cross-girder.

Fig. 31 shows a cross-section of the structure at the platforms. The ends of the express-track stringers which rest on top of the bent, are cut so as to form seats for two 24-in. I-beams which are flush at the top with the top of the stringers, and cantilever at each end, the cantilever ends supporting the inside girders which carry the express platform. The outside edge of the platform was supported on girders resting on columns which were again supported by the lower-level cross-girders. In order to make the vertical distance from the lower deck to the upper-deck platform as small as possible, the express platform construction was made very shallow. On account of the additional loading, due to the express platforms, the lower-deck cross-girders and columns were renewed.

At 42d Street there is a branch leading to the New York Central Terminal Station, to which passengers transfer from the Third Avenue Line Station at 42d Street. Before the reconstruction, access to this branch from the up-town platform at 42d Street was had by a bridge crossing over the tracks to the down-town platform, which connected directly with the branch platform. The overhead bridge interfered with the new overhead express station structure and therefore was removed, and in its place a bridge was provided under the lower-deck structure. On account of the small head-room between the street and the structure, the depth of the stringers, under which the bridge had to pass, was cut down, as shown by Fig. 29. The old stringers were lattice girders. The remodeled portions of the stringers were reinforced with web-plates.

Details of Design.—Section No. 4-A.—Section No. 4-A comprised the Second Avenue Line from Chatham Square to 116th Street. Prior to this reconstruction work, a center-track construction existed on certain portions of the line. The work comprised the completion of the center-track structure; the double latticing of the existing center-track stringers; the reconstruction of the stations at 14th, 42d, and 86th Streets for express service; and the rebuilding of the island station at 92d Street.

Fig. 30 shows a cross-section of the structure before the reconstruction. The columns are of the six-section Phœnix type. The cross-

girders are twin lattice girders and the track stringers are lattice girders supported on top of the cross-girders by a lip 7 in. deep.

Where the work consisted of adding center-track girders only, no changes were made to the existing structure. As it was generally desirable to have the center track at the same elevation as the outside tracks, and necessary at switches and cross-overs, the new center-track stringers were also designed with a 7-in. lip, as shown by Fig. 34. The effective lip consisted of the top flange angles, the bottom angles of the lip, which were extended about 4 ft. over the main portion of the girder, and two 6-in. ship-building channels, also extended 4 ft. over the main portion of the girder. The end shear is 36 000 lb. for

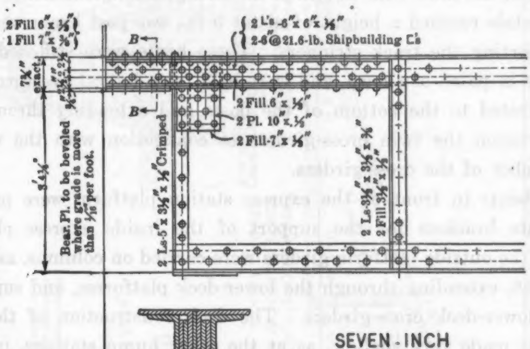


FIG. 34.

a 45-ft. span, which is the standard length, and the section modulus of the combined lip section is 58.6. Assuming the load on the lip to be applied 12 in. from the main portion of the girder, the maximum stress in the lip is 7 500 lb. This, however, is only true if the rivets connecting the different sections of which the lip is constructed are sufficient to make them act together as a unit. The rivets in the web portion of the lip were not sufficient for this purpose, as they were too close to the neutral axis. The channel section, therefore, was used, in order to be able to utilize the rivets in the horizontal flanges to take up the shear, and as the flanges in a standard 6-in. channel were too narrow to use $\frac{7}{8}$-in. rivets, the ship-building channel was selected.

As the head-room under the structure at the 14th Street Station did not permit the construction of a mezzanine station, the "hump" type of express station was used. At this station, prior to the reconstruction, there existed center-track girders, but they were removed, as they interfered with the construction of the new bents, and were not used in the new structure, as the 7-in. lip construction of these girders was not desirable in the new work. Fig. 35 shows the details of the new structure carrying the express track. Lips of increasing depth were used to raise the track stringers to the new grade at the beginning of the approach. When the stringers reached above the cross-girders, pedestals, riveted to the ends of the stringers, were used, being intended at the same time to stiffen the stringers longitudinally. When the pedestals reached a height of about 5 ft., two-post bents were used for supporting the track stringers. These bents were stiffened longitudinally in pairs, as shown, and were held from lateral overturning by plates riveted to the bottom of the posts and extending through the space between the twin cross-girders to connection with the vertical web member of the cross-girders.

The bents in front of the express station platform were provided with plate brackets for the support of the inside express platform girders; the outside platform girders were carried on columns, as shown by Fig. 36, extending through the lower-deck platforms, and supported on the lower-deck cross-girders. The deck construction of the platform was made very shallow, as at the other hump stations, in order to keep the level of the upper deck as low as possible.

The 86th Street Station was also a hump station. In the original scheme, no express station was to be built at this location, but, after all the steel for the new track stringers, which were designed with 7-in. lips, to be set directly on top of the existing cross-girder, had been fabricated and delivered, the Public Service Commission ordered an express station to be built there. The general details of the steel structure were similar to those at 14th Street, except that seats were provided on top of the supporting bents to carry the lip ends of the track stringers as fabricated.

The 42d Street Station had originally two side-platforms. As the head-room from the street to the under side of the structure could be made sufficient to place a mezzanine station by raising the existing track construction about 1 ft., this type of station was decided on

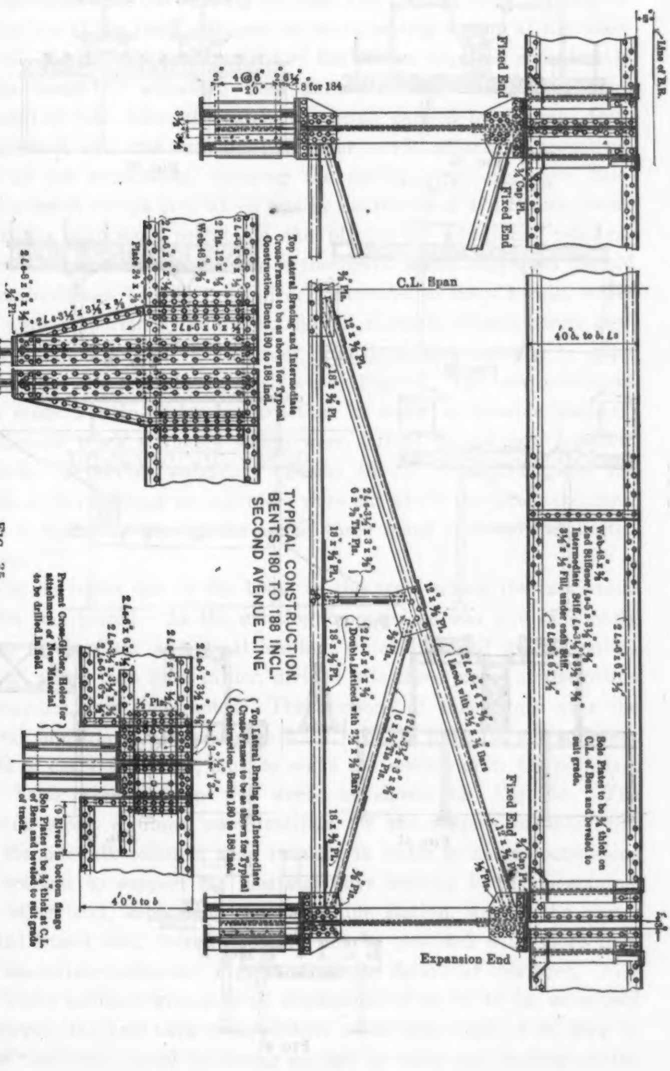

TYPICAL CONSTRUCTION
BENTS 180 TO 188 INCL.
SECOND AVENUE LINE
Line of R.R.
Fixed End
Fixed End
C.L. Span
Fixed End
Expansion End

FIG. 25.

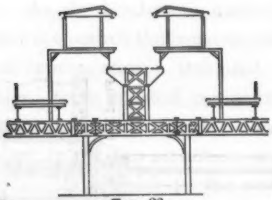


FIG. 36

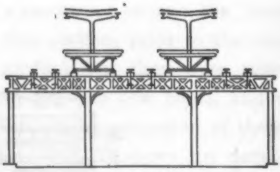


FIG. 37

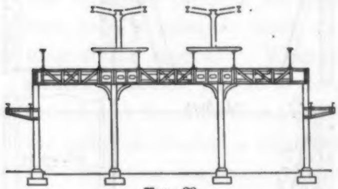


FIG. 38

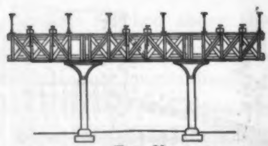


FIG. 39

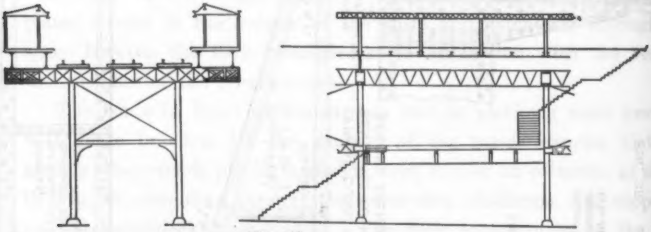


FIG. 40

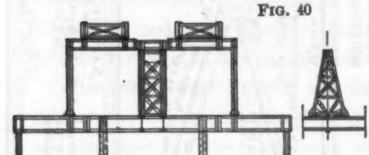


FIG. 41

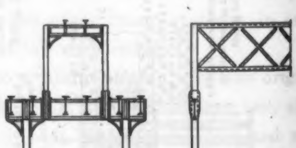


FIG. 42

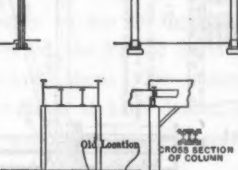


FIG. 43

at this point. The raising of the track structure was accomplished, without disturbing the existing columns and cross-girders, by supporting the lips of the track stringers on seats, resting on top of the cross-girders. As the new arrangement of the station required considerably greater width of structure than formerly, the cross-girders were extended at both sides, the extensions being carried by columns from the sidewalk near the curb line; in other words, after the reconstruction, all the cross-bents, carrying the station structure, were four-column bents, except one, which was in the center of 42d Street, where the traffic conditions prohibited the placing of additional columns. Before the reconstruction, a few of the bents, which supported the old station buildings, had the cross-girders extended to the sidewalk, where they were supported by columns; the old sidewalk columns were generally retained, but the cross-girder extensions were renewed in order to carry the new track loads, as shown by Fig. 37. The new extensions were made a twin-girder construction in order to provide supports for the old track stringers which were shifted to the new location. In order to prevent secondary stresses in the cross-girder, the top chords of the existing cross-girders were cut above the street columns, so as to make the cross-girder act as three spans of simply supported girders.

Fig. 38 shows one of the bents in the span, where the mezzanine station was placed. As the existing cross-girder was not sufficiently strong to carry the additional loading, which included the mezzanine station, a new twin cross-girder, divided into three pieces at the points of supports, was constructed. The portions of this girder over the central supports were made with solid webs, in order to get sufficient rivets to carry the shear, and, to make the section open for painting, etc., holes were cut in the webs, as shown on Fig. 38. The existing street columns were retained for the support of this bent, but the sidewalk columns were renewed in order to make connections for brackets to support the new stairways leading to the mezzanine. The other bent, supporting the mezzanine station, was in the center of 42d Street and, therefore, could not be provided with more than two supporting columns. Fig. 39 shows the details of this bent. New and heavy columns with over-all dimensions of 15 by 18 in. were used to support the new twin cross-girders, which were made 6 ft. deep in order that they would be strong enough to carry the loading on the

cantilevers, 16 ft. 6 in. long. The track was carried on through bridge girders, which also supported the platforms and the suspended mezzanine station.

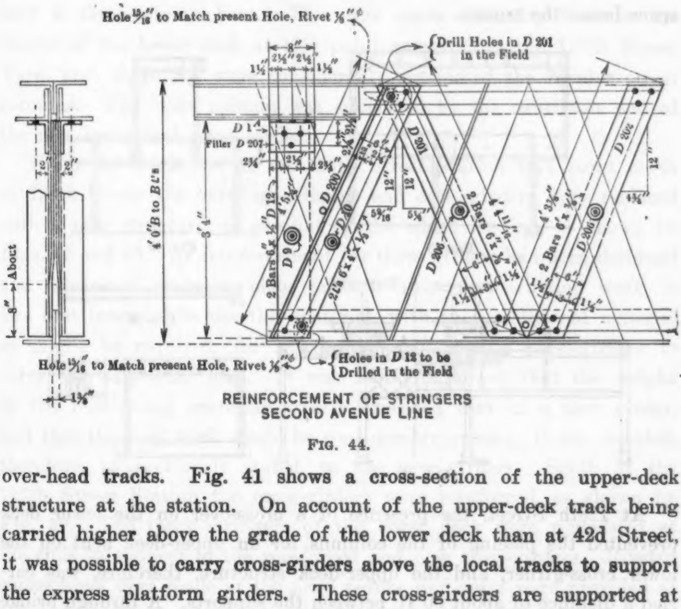
At 92d Street the structure is high above the street. The old station had a center platform, and the station building was at platform level. Access to the station was obtained by stairs at both sides of the street. The stairs were connected by a bridge under the track structures, and a stairway led from this bridge to the station building. As the station building and the center platform were in the way of the new center track, the station was reconstructed with side platforms and a mezzanine station building, before the existing station structure was removed. Fig. 40 shows the construction. The twin cross-girders were lengthened to support the new platform girders, and the latter were set on top of the cross-girders, and carried the new platform construction. For the support of the mezzanine station, two girders were framed into the Phœnix columns supporting the structure, one on each side of the street in the span, when the mezzanine station was built. As it was not considered advisable to disturb the existing riveting of the Phœnix columns, beveled fillers, with holes cut at the locations of the rivet heads, were added between the flange of the column and the splice-plates, connecting the girders to the column.

The purpose of double latticing the existing center-track stringers was to reduce the bending stress in the top flanges by shortening the distance between the panel points, and to reinforce the connection at the end lip, which, experience has shown, is the weakest point of the stringer. The general method of reinforcing is shown by Fig. 44, and is the standard reinforcement for track stringers of this type used prior to this work. The method of end reinforcing is not entirely satisfactory, as the rivets are in tension, but, nevertheless, it acts as an additional safeguard.

Details of Design.—Section No. 5-A.—Section No. 5-A included the Third Avenue Line between 116th and 129th Streets and in 129th Street between Third and Second Avenues. The work involved the reconstruction of the station at 125th Street for express service and the connection of the express track with the upper deck of the Harlem River Bridge. North of 125th Street Station it was desired to retain the existing center track on the lower level, as it was used as the main north-bound track, while the outside track was used as an

approach track to the Company's Yard at 129th Street. It became necessary, therefore, to carry the over-head track high enough to provide sufficient head-room for the trains below, and as it was desired to keep the over-head track approximately level through the station, this height was maintained to the south end of the over-head platforms.

The approach to the over-head track commenced at a point near 121st Street. As there had been a center-track structure prior to the reconstruction, the general type of construction was similar to that described under Section No. 3, for the northerly approach to the 42d Street Station, with tapered girders, seats, and towers supporting the



REINFORCEMENT OF STRINGERS
SECOND AVENUE LINE

FIG. 44.

over-head tracks. Fig. 41 shows a cross-section of the upper-deck structure at the station. On account of the upper-deck track being carried higher above the grade of the lower deck than at 42d Street, it was possible to carry cross-girders above the local tracks to support the express platform girders. These cross-girders are supported at the inside by the towers carrying the track structure and at the outside by columns resting on the lower-deck cross-girders.

North of the 125th Street Station, where the lower-deck center track was retained, the distance between the tracks was increased by moving the supporting track stringers sidewise with the tracks to make room for the columns of two-column bents, set on the top of the lower-deck cross-girders to carry the upper-deck track structure.

Fig. 42 shows the details of these bents. The column section was made only 12 in. wide, in order to make the sidewise shifting of the tracks as small as possible. The columns were spliced to the supporting lower-deck cross-girders, and extended to the top of the upper-deck cross-girders, which were riveted to the column shafts. The upper-deck track stringers were of the lip type, resting on top of the cross-girders, and the bottoms of the stringers were flush with the bottoms of the cross-girders. Longitudinally, the bents were provided with stiffening trusses extending from the top of the columns to within about 6 ft. of the lower track level, so as to leave a clear walking space below the trusses.

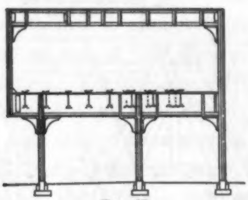


Fig. 45

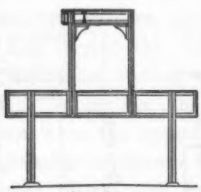


Fig. 46

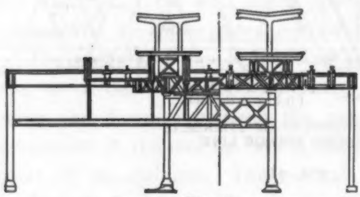


Fig. 47

Fig. 48

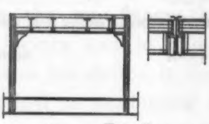


Fig. 49

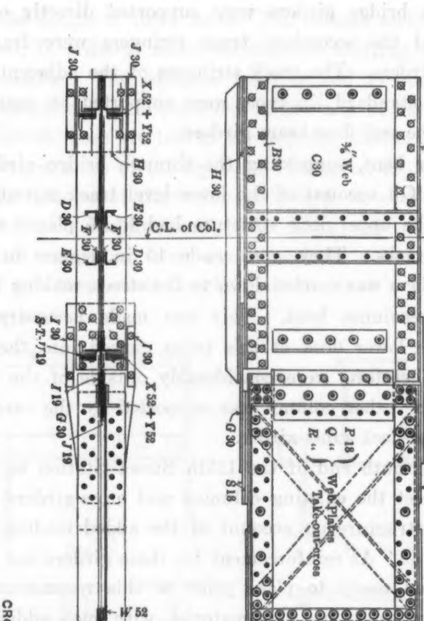
At 128th Street, the presence of a cross-over on the lower deck prevented the placing of the columns for an upper-deck bent on the lower cross-girder, and the upper-deck structure, therefore, was carried a distance of about 90 ft. between the supports. A through bridge construction was used for this span, as shown by Fig. 43, in order to get sufficient depth of the longitudinal girders. Fig. 43 shows also a cross-section of one of the supporting bents. As the column section could not be materially widened, on account of the necessity of maintaining the proper clearance between the side of the car and the column, the column section necessary for the increased vertical loading and wind stresses was obtained by increasing the column shaft lengthwise.

The through bridge girders were supported directly on top of the columns, and the secondary track stringers were framed into the floor-beam girders. The track stringers of the adjacent spans, which were of the standard lip type, were supported on seats attached to the web of the end floor-beam girders.

The other bent, supporting the through bridge girders, is shown by Fig. 45. On account of the lower-level track layout, the columns supporting the upper-deck structure had to be placed outside of the lower-deck tracks. They were made 15 in. square in cross-section, and one of them was carried down to the street, making the lower-deck bent a three-column bent. This was made necessary because the tracks of the lower deck at this point curved into the 129th Street Yard, and, therefore, were considerably outside of the regular street columns. The other column was supported by the cantilever end of the new lower-deck cross-girder.

From the south end of the 125th Street Station to a point north of 128th Street the existing columns and cross-girders were replaced with a new structure on account of the added loading, as shown by Figs. 42 and 45. As reinforcement for these girders had been designed and fabricated ready to place prior to this reconstruction work, it was first intended to use this material, with such additional material as might be required, for reinforcing the existing cross-girders to carry the additional load. It was found, however, that the weight of the reinforcing material greatly exceeded that of a new girder, and that the field work would be considerably greater; it was decided, therefore, as previously stated, to use new girders. South of the 125th Street Station the cross-girders were reinforced, as shown by Fig. 50. The track stringers of the lower deck were remodeled at the ends to connect to the reinforced and new cross-girders. Riveted connections were made at the fixed points, and seats were provided at the expansion points, as shown by Fig. 50.

North of the 125th Street Station, where the outside track girders were shifted, the procedure was first to erect the new cross-girders with the stringers in the original position, and then to shift the stringers to their new position. The cross-girders were provided, therefore, with temporary seats for the girders in the old position, as well as with permanent seats.



CROSS-GIRDERS-992 TO 1001 INCL. THIRD AVENUE LINE

FIG. 50.

The existing structure in 129th Street was sufficiently strong to carry the additional loading of the upper level track without reinforcement. At the westerly end of 129th Street, the existing structure carried a wide platform of the 129th Street Station. The columns supporting the upper-track structure were carried through this platform to the lower-deck cross-girders, to which they were spliced. East of the platform the column locations were determined by the track arrangement of the lower deck. The general details of the bents were similar to those of the bents in Third Avenue, north of the 125th Street Station, except that the stringers were supported on seats attached to the web of the cross-girders and that the latter, in several cases, cantilevered at the one end over the column and supported the one line of track stringers on the cantilever end, as shown by Fig. 46. At the east end of the structure, the new track divided into two to connect with the upper-deck tracks of the Harlem River Bridge.

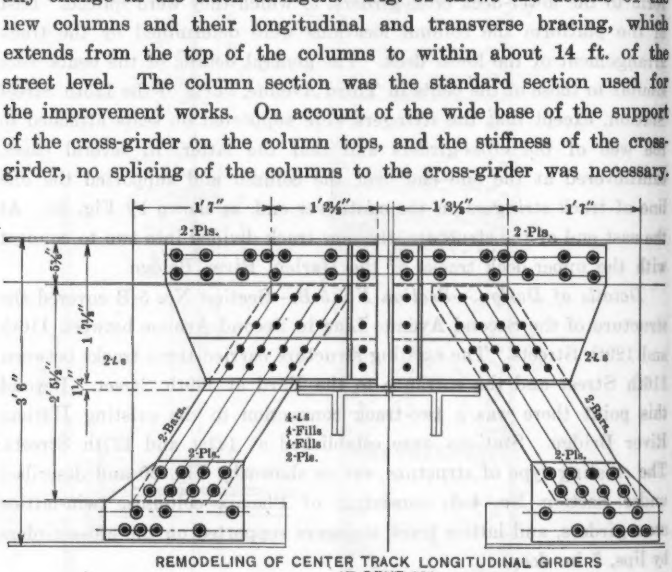
Details of Design.—Section No. 5-B.—Section No. 5-B covered the structure of the Second Avenue Line in Second Avenue between 116th and 129th Streets. The existing structure carried three tracks between 116th Street and the entrance to the Yard at 129th Street. Beyond this point there was a two-track connection to the existing Harlem River Bridge. Stations were established at 121st and 127th Streets. The existing type of structure was as shown by Fig. 30 and described under Section No. 4-A, consisting of Phœnix columns, twin-lattice cross-girders, and lattice track stringers supported on the cross-girders by lips, 7 in. deep.

The new plan involved replacing the existing station at 127th Street with a new express station at 125th Street, the connection of the express and local tracks with the upper deck of the new Harlem River Bridge, and the connection of the reconstructed local tracks with the lower deck of the Harlem River Bridge and with the Yard.

The existing structure at 125th Street was low, but, in order to make proper connection to the upper deck of the Harlem River Bridge, it was raised sufficiently to make it possible to construct the new station at 125th Street as a mezzanine station. All three tracks were raised, starting at a point near 122d Street, and, at 125th Street, the rise amounted to about 7½ ft.

Fig. 51 shows the construction used to raise the track at the beginning of the grade. The ends of the track stringers were remodeled,

and the sizes of the supporting lips were gradually increased until a point was reached where the track stringers rested entirely on top of the cross-girders. After this point was reached the existing cross-girders were raised with the track stringers, which necessitated replacing the existing columns with new and longer ones, but did not involve any change in the track stringers. Fig. 52 shows the details of the new columns and their longitudinal and transverse bracing, which extends from the top of the columns to within about 14 ft. of the street level. The column section was the standard section used for the improvement works. On account of the wide base of the support of the cross-girder on the column tops, and the stiffness of the cross-girder, no splicing of the columns to the cross-girder was necessary.



REMODELING OF CENTER TRACK LONGITUDINAL GIRDERS AT BENT 892

FIG. 51.

As the columns had to be erected under cross-girders in place and over existing foundations with anchor-bolts in place, a detachable type of base was used, but, on account of the sub-surface conditions, they differed somewhat from those already described. There was a 12-in. gas pipe and also a 12-in. water main at the west side of Second Avenue, almost directly under the columns. The old foundations, which were retained, were divided at the top into three parts, with the gas and water mains passing between the three portions of the foundations. The old columns had cast-iron bases, into which the column shafts fitted and were connected by rust caulking. Some of the cast-iron bases were shaped to bridge across the openings in the foundations,

and others rested on top of grillage beams spanning the openings. The detachable parts of the new bases were made of a similar shape to fit the anchor-bolts, but they were of cast steel, and were connected to the fixed portion of the base by six bolts. The bases on the east side of the street, where the foundations were built in one piece, were made shorter, but, for the sake of uniformity, of a similar construction.

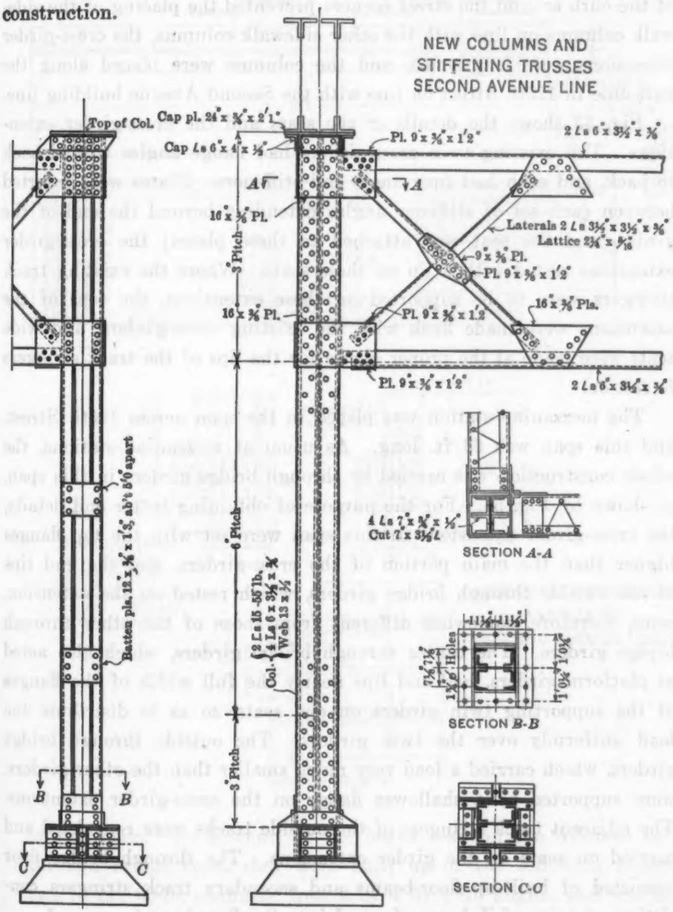


FIG. 52.

The width of the structure had to be increased throughout the 125th Street Station on account of the additional space needed for the platforms. This was accomplished by adding new pieces to both ends of the cross-girders, supported at one end by seats attached to the existing cross-girders and at the other end by columns on the sidewalk, near the curb line. At 125th Street, where the curvature of the curb around the street corners prevented the placing of the sidewalk columns on line with the other sidewalk columns, the cross-girder extensions were lengthened, and the columns were placed along the curb line in 125th Street on line with the Second Avenue building line.

Fig. 53 shows the details of the seats and the cross-girder extensions. The existing twin cross-girders had flange angles turned back to back, and each had four angle end stiffeners. Plates were inserted between each set of stiffener angles extending beyond the end of the girder, and the seat was attached to these plates; the cross-girder extensions rested with a lip on these seats. Where the existing track stringers were to be supported on these extensions, the tops of the extensions were made flush with the existing cross-girders, and wide seats were made at the proper places for the lips of the track stringers to rest on.

The mezzanine station was placed in the span across 125th Street, and this span was 63 ft. long. As usual at mezzanine stations, the whole construction was carried by through bridge girders in this span, as shown by Fig. 54. For the purpose of obtaining better end details, the cross-girder extensions in this span were set with the top flanges higher than the main portion of the cross-girders, and the end lips of the outside through bridge girders, which rested on the extension, were, therefore, somewhat different from those of the other through bridge girders. The center through bridge girders, which also acted as platform girders, had end lips nearly the full width of the flanges of the supporting twin girders on end seats, so as to distribute the load uniformly over the twin girders. The outside through bridge girders, which carried a load very much smaller than the other girders, were supported by a shallower flange on the cross-girder extensions. The adjacent track stringers of the outside tracks were remodeled and carried on seats on the girder extensions. The through bridge floor consisted of built-up floor-beams and secondary track stringers consisting of pairs of I-beams framed into the floor-beams.

Outside of the mezzanine span, the platform stringers consisted of plate girders supported on top of the cross-girders. Fig. 47 shows a cross-section of the structure at the through-bridge span and outside

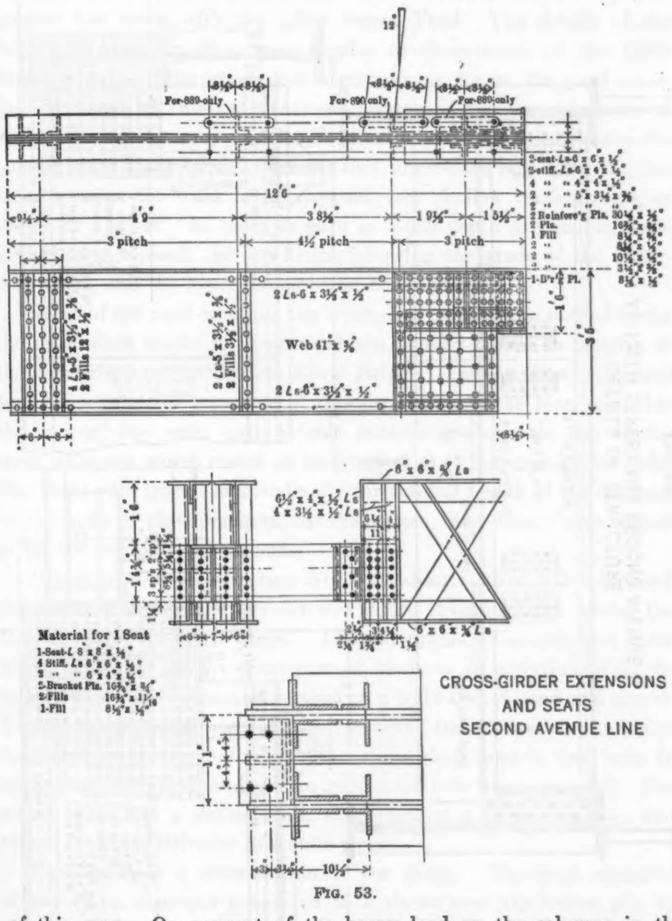
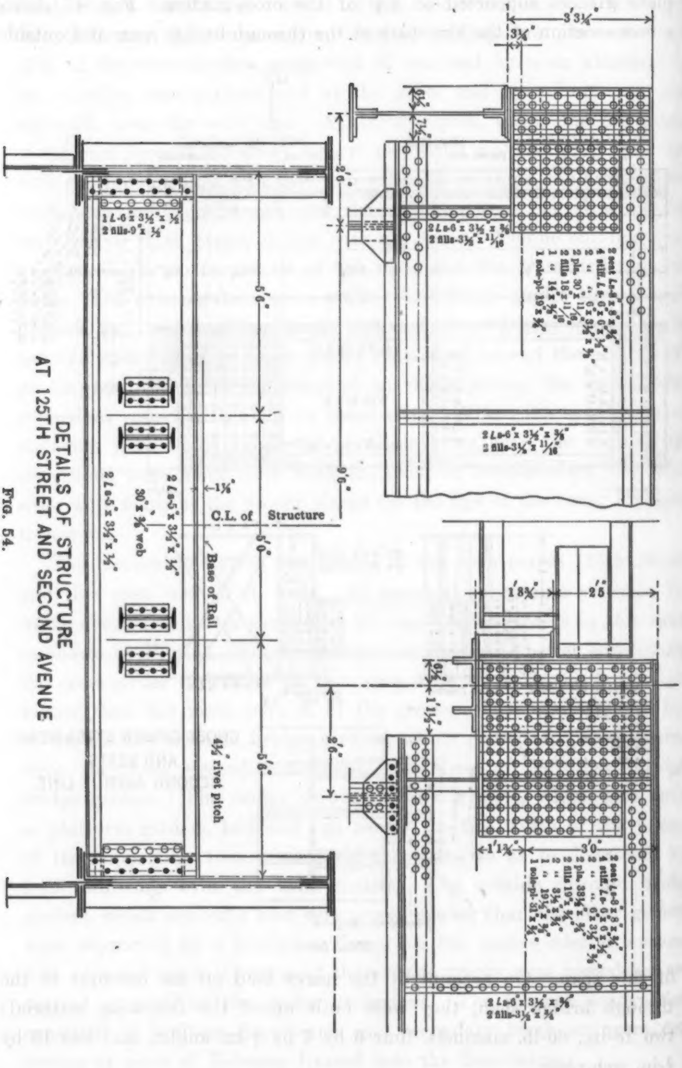


FIG. 53.

of this span. On account of the heavy load on the columns in the through bridge span, they were built up of the following material: two 18-in., 60-lb. channels, four 6 by 4 by $\frac{3}{4}$-in. angles, and one 13 by $\frac{3}{4}$-in. web-plate.



DETAILS OF STRUCTURE
AT 125TH STREET AND SECOND AVENUE

FIG. 54.

North of the 125th Street Station the three-track line was changed into four tracks, with the two center tracks ascending to the level of the upper deck of the Harlem River Bridge and the two outside tracks descending to the original level just south of 128th Street where connection was made with the 129th Street Yard. The details of the lower-deck reconstruction were similar to those south of the 125th Street Station. The upper-deck structure, as far as the yard crossing, consisted of two-post bents set on top of the lower-deck cross-girders, braced laterally and longitudinally. At one point, where the outside lower track turns under the new upper-deck structure, the distance between the track levels required very shallow track girders, as shown by Fig. 48. In order to gain as much depth as possible, these girders were tapered, the top flange following the grade of the upper-deck track, and the lower flange being horizontal.

North of the yard crossing, the lower-deck tracks were carried under the upper-deck tracks in order to obtain as easy curves as possible at the connection to the Harlem River Bridge. Fig. 49 shows a typical bent supporting the structure at this location. As the span was wide the cross-girders were made deeper, extending above the top of the track stringers, which rested on seats attached to the cross-girder webs. The bents were braced longitudinally for the full length of the columns and laterally by plate brackets. At the bottom, the columns were spliced to the supporting cross-girders.

Details of Design.—Section No. 5-C.—Section No. 5-C comprised the bridge carrying the Second and Third Avenue Lines across the Harlem River at 129th Street. The old bridge (Fig. 56) had three spans, supported on five piers carried down to an elevation of 32 ft. below mean high water, and resting on a solid bed of sand and gravel. The old bridge, which carried two tracks and two footwalks, was a pin-connected structure, built in 1887; since that time it has been in continuous operation, and was in good condition when removed. The center span was a swing span, supported on a drum bearing, and turned by a rope drive by hydraulic power.

Fig. 55 shows a cross-section of the drum. The drum consisted of two 12-in. channels connected to a sleeve over the center pin by shallow built-up girders. It supported the bridge structure at six points, and was itself supported on rollers running on a circular track and held in position by tie-rods and angles. To the outside

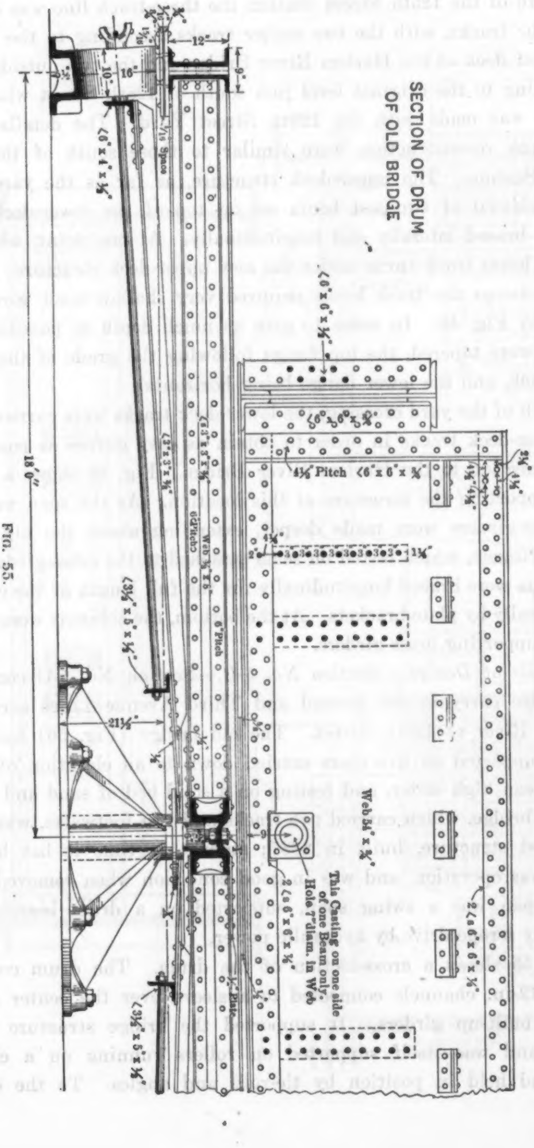

SECTION OF DRUM
OF OLD BRIDGE
18' 0"
21½"
12"
16"
4½" Pitch
This Casting on one side
of one Beam only

FIG. 55.

ends of the axles of the rollers was attached a ring with a semicircular groove in which was laid the cable that turned the bridge. This cable went around about three-quarters of the ring, and was carried up to the under-side of the operating tower over vertical sheaves, set tangential to the ring. The cable was then carried over sheaves that changed the direction to the horizontal, so that the cable could be connected to the horizontal hydraulic ram under the floor of the operating tower. It will be noted that the speed of the cable determines the horizontal speed of the rollers, but that the speed of the drum is determined by the circumferential speed of the rollers, and the drum, therefore, travels at a speed equal to twice that of the cable. It may be stated that the arrangement worked satisfactorily, and that the bridge could be opened very quickly, but that, on account of the shallow drum, the bearing on the rollers was not uniform and caused them to wear out.

The new track facilities, on account of the Manhattan Elevated Improvements, involved the construction of a double-deck four-track structure in place of the existing two-track bridge, but it was necessary to maintain traffic, except during short periods, over the two existing tracks. It was decided, therefore, to build an entirely new bridge at a location where the work would not interfere with the operation of the existing bridge, and, when the erection of the new bridge was finished, to replace the old spans with the new spans complete. The method of doing this work is described later under the heading, "Erection."

Fig. 57 shows the new bridge. The length of the end spans, from center to center of piers, was 103.28 ft., and the length of the swing span was 248.4 ft. over all. The clear openings of the swing span were 103.7 ft. The new bridge was built as a pin-connected structure. Fig. 86 shows the construction of the north span.

The new swing span was center-supported. It was at first intended to design the new bridge as rim-supported, similar to the old bridge, but it was found that the depth of the new drum, needed in order to avoid undue deflection, would necessitate the removal of a considerable portion of the center pier, and this could not be done without interfering with the operation of the bridge for a considerable length of time. The casting for the center support of the new bridge, as designed, necessitated the removal of a portion of the pier, but this

work could be done without interfering with the traffic, as hereafter described.

Plate IX shows the center casting and the arrangement of the operating machinery. The weight of the bridge was carried to the casting through three disks, on which the bridge turned. The center disk was made of phosphor-bronze and the other two were steel forgings.

The turning machinery was driven by two 40-h.p. electric motors, and two other 40-h.p. electric motors were provided for the operation of wedges and rail-lifts. After the machinery had been received and erected, and while the swing span was still resting on the pile foundation on which it was erected, an attempt was made to work the operating machinery. It was then found that the bearings for the beveled gears were not sufficiently stiff under the stress, and that the gears were thrown out of line and became bound. The cause of the trouble was that the bearings for the beveled gears were cast and supported independently of one another. A new set of castings was designed, therefore, with two bearings for the corresponding gears in the same casting, and after they had been placed no further trouble was experienced.

The method of determining the live-load stress in the swing span deserves to be stated. When closed, the bridge is supported on three bearings, and is, therefore, statically indeterminate. The usual way of determining the stress, by the three-moment method, which assumes uniform moment of inertia throughout the truss, was evidently not satisfactory, as the moment of inertia varies greatly, and it was found by comparing the stresses obtained in this manner with those determined by the accurate method hereafter described, that the stresses by the first method were in error as much as 100 per cent.

The method used was developed for this purpose by F. W. Gardiner, M. Am. Soc. C. E., and is described as follows:

Consider the member, *H-m* (Fig. 58), cut at the center; the truss will then be two single spans, and will be statically determinate. Apply to the cut ends of *H-m* two equal and opposite forces just sufficient to hold the ends in contact. The stresses in the truss members for any condition of loading will then be the combined stresses computed as a single span and the stresses induced by a pair of equal and opposite

FIG. 56.—OLD HARLEM RIVER BRIDGE.

FIG. 57.—HARLEM RIVER BRIDGE. NEW SWING SPAN.

PLATE IX.
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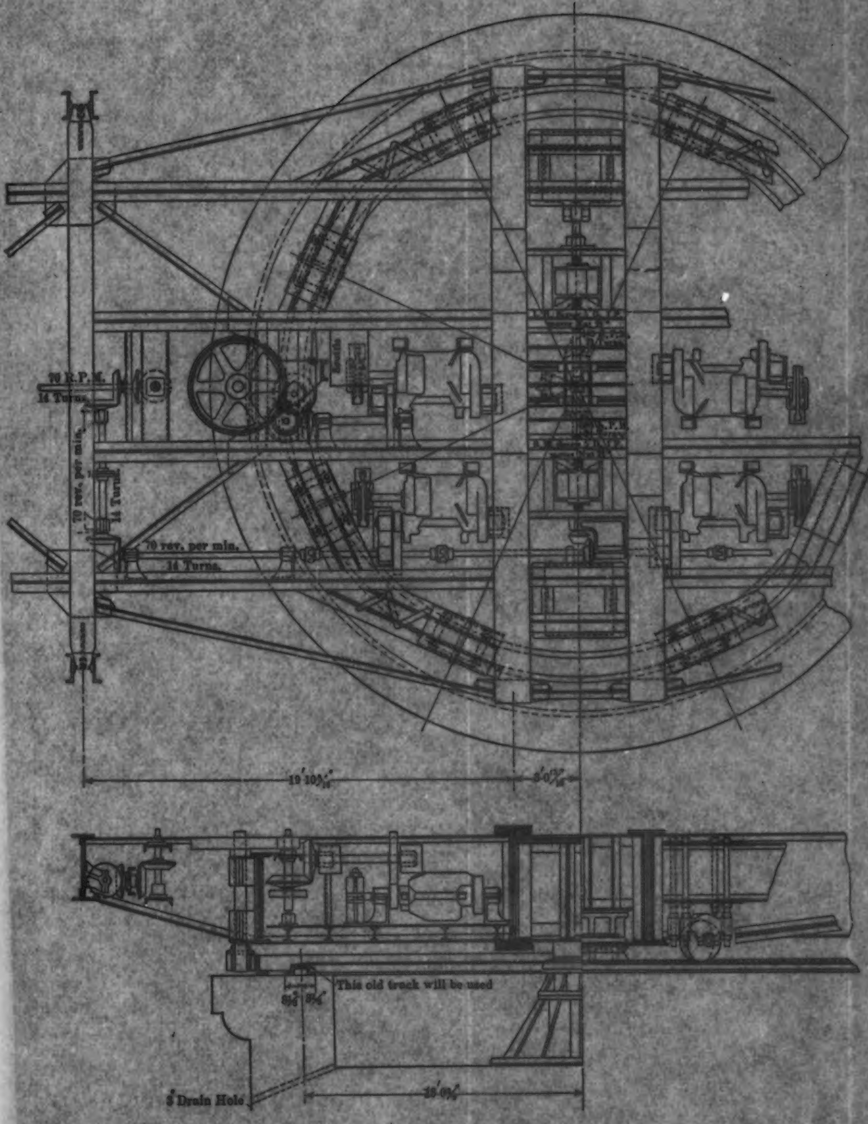
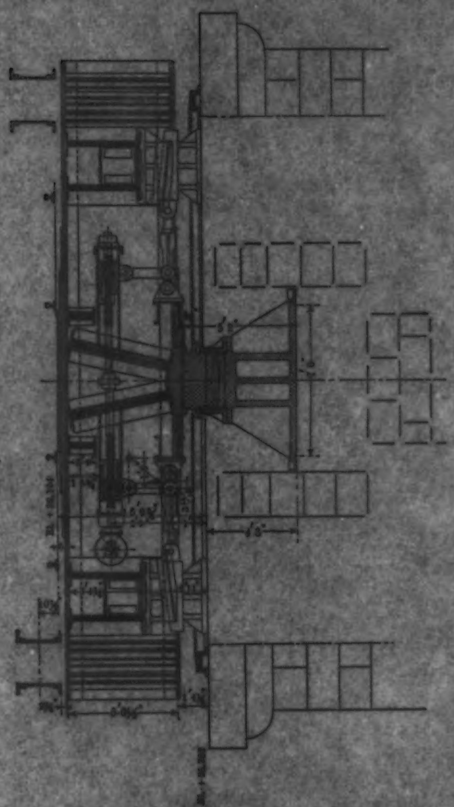
GARDINER AND JOHANNESSON ON
ELEVATED RAILWAY IMPROVEMENTS.





PLAN AND SECTIONS SHOWING
CARRYING GIRDERS, PIVOT, AND OPERATING MACHINERY
AT CENTER PIER OF SWING SPAN.

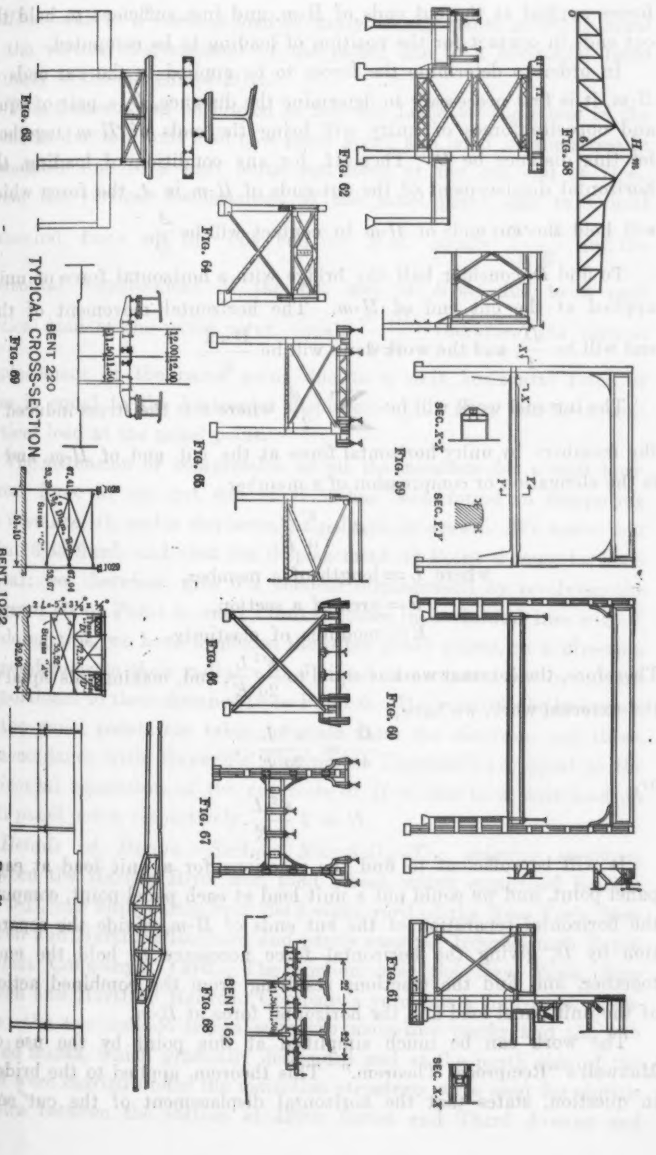


FIG. 58 FIG. 59 FIG. 60 FIG. 61 FIG. 62 FIG. 63 FIG. 64 FIG. 65 FIG. 66 FIG. 67 FIG. 68 FIG. 69 FIG. 70 FIG. 71

forces applied at the cut ends of *H-m*, and just sufficient to hold the cut ends in contact for the position of loading to be computed.

In order to determine the forces to be applied to the cut ends of *H-m*, it is first neecessary to determine the distance, D, a pair of equal and opposite forces of unity will bring the ends of *H-m* together; let this distance be D. Then, if, for any condition of loading, the horizontal displacement of the cut ends of *H-m* is $\varDelta$, the force which will hold the cut ends of *H-m* in contact will be $\frac{\varDelta}{D}$.

To find D, consider half the bridge with a horizontal force of unity applied at the cut end of *H-m*. The horizontal movement of this end will be $\frac{D}{2}$, and the work done will be $\frac{D \; x \; 1}{4}$.

The internal work will be $\sum \frac{s' \; d}{2}$, where s' is the stress induced in the members by unity horizontal force at the cut end of *H-m*, and d is the elongation or compression of a member;

$$d = \frac{s' \; L}{a \; E},$$

where $L =$ length of a member,
$a =$ area of a section,
$E =$ modulus of elasticity.

Therefore, the internal work is equal to $\frac{s'^2 \; L}{2a \; E}$, and, making this equal to the external work, we have,

$$\frac{D}{4} = \frac{s'^2 \; L}{2a \; E},$$

or,

$$D = 2 \, \frac{s'^2 \; L}{a \; E}.$$

It will be sufficient to find the reactions for a unit load at each panel point, and we could put a unit load at each panel point, compute the horizontal separation of the cut ends of *H-m*, divide the separation by D, giving the horizontal force necessary to hold the ends together, and find the reactions resulting from the combined action of the unit panel load and the horizontal force at *H-m*.

The work can be much simplified at this point by the use of Maxwell's "Reciprocal Theorem." This theorem, applied to the bridge in question, states that the horizontal displacement of the cut end

of *H-m*, due to a vertical load of unity at any panel point, is equal to the vertical displacement of the panel, due to a horizontal force of unity at the cut end of *H-m*.

This theorem can be proved as follows: Let S = the stress in the members due to a unit load at a panel point, and let S' = the stress in a member due to a unit horizontal force at the cut end of *H-m*. Then, the vertical displacement of the panel point, due to a unit horizontal force at the cut end of *H-m*, equals $\frac{SS'\,L}{a\,E}$, and the horizontal displacement of the cut end of *H-m*, due to a unit vertical load at the panel point, equals $\frac{S'\,SL}{a\,E}$, therefore, the vertical displacement of the panel point due to a unit horizontal force at *H-m* is equal to the horizontal displacement of *H-m* due to a unit vertical load at the panel point.

The extension or compression of all the members for a unit horizontal force at the cut end of *H-m* has been found in computing the distance, *D*, and a displacement polygon is drawn. We know that Point 6 is fixed, and that the displacement of Point 0 must be horizontal; we, therefore, give 0 a vertical displacement by revolving the diagram about Point 6, until Point 0 comes in horizontal line with 0. In doing this, we have displaced the other panel points, in a direction perpendicular to their radius with Point 6 as a center, and in amount proportional to their distance from Point 6. The vertical displacements of the panel points are taken by scale from the diagram, and these, in accordance with Maxwell's "Reciprocal Theorem", are equal to the horizontal separation of the cut ends of *H-m* due to a unit load on each panel point, respectively.

Details of Design.—Section No. 5-D.—The existing structure between the Harlem River and 133d Street, which comprised Section No. 5-D, was built partly over the freight yard of the New York, New Haven and Hartford Railroad and partly over the Interborough Rapid Transit Company's Yard. The portion over the New York, New Haven and Hartford Railroad Company's yard was a four-track structure; the two outside tracks were the main-line tracks and the two inside tracks, which gradually descended and at the north side of the yard were carried under the main-line structure, were used for shuttle service between the station at 129th Street and Third Avenue and

the New York, New Haven and Hartford Railroad Passenger Station at Willis Avenue. North of the Railroad Company's yard the existing structure carried only the two main tracks, and through the yard the existing columns were placed so as not to interfere with the tracks and platforms.

The new structure consisted of two additional tracks connecting with the upper deck of the Harlem River Bridge. Through the New York, New Haven and Hartford Railroad Company's Yard they were directly above the shuttle tracks, and north of the yard they were continued between and above the outside main tracks. Fig. 72 shows the general plan and profiles through this section. It may be noted here that the curved track layout and the skew location of the bents throughout this section necessitated a specially careful survey of the existing structure, and involved very extensive calculations to determine the lengths and locations of the members forming part of the new structure. It was found that the work was much facilitated and more easily checked by using analytical geometry in the calculations. A system of rectangular co-ordinates was used; the zero point and the location of the axes were chosen arbitrarily, but in such a manner that all the co-ordinates were positive, but as small as possible, so as not to have unnecessarily large figures to compute. The equations of all the cross-girders of the old structure were then calculated, as well as those of the new tracks, cross-girders, etc.; also, the co-ordinates of the points of intersection, determining the lengths and locations of the new structure members. This method of calculation was very effective, and had the important advantage that errors were not carried from point to point, and when discovered were easily corrected.

Plate X shows the alignment of the old and new structures through the New York, New Haven and Hartford Railroad Company's Yard. The first bent of the old structure was supported on a masonry pier. The columns, supporting the new structure, were placed one at each end of the old pier, as the track spacing of the lower deck did not allow sufficient room between the tracks for the supporting columns. In order to brace laterally the column shafts, which were more than 48 ft. long, a yoke, consisting of 15-in. channels, was fastened to the columns at the level of the cap-stone of the pier, and secured to the cap-stone by expansion bolts, as shown by Fig. 59. The columns

PLATE X.
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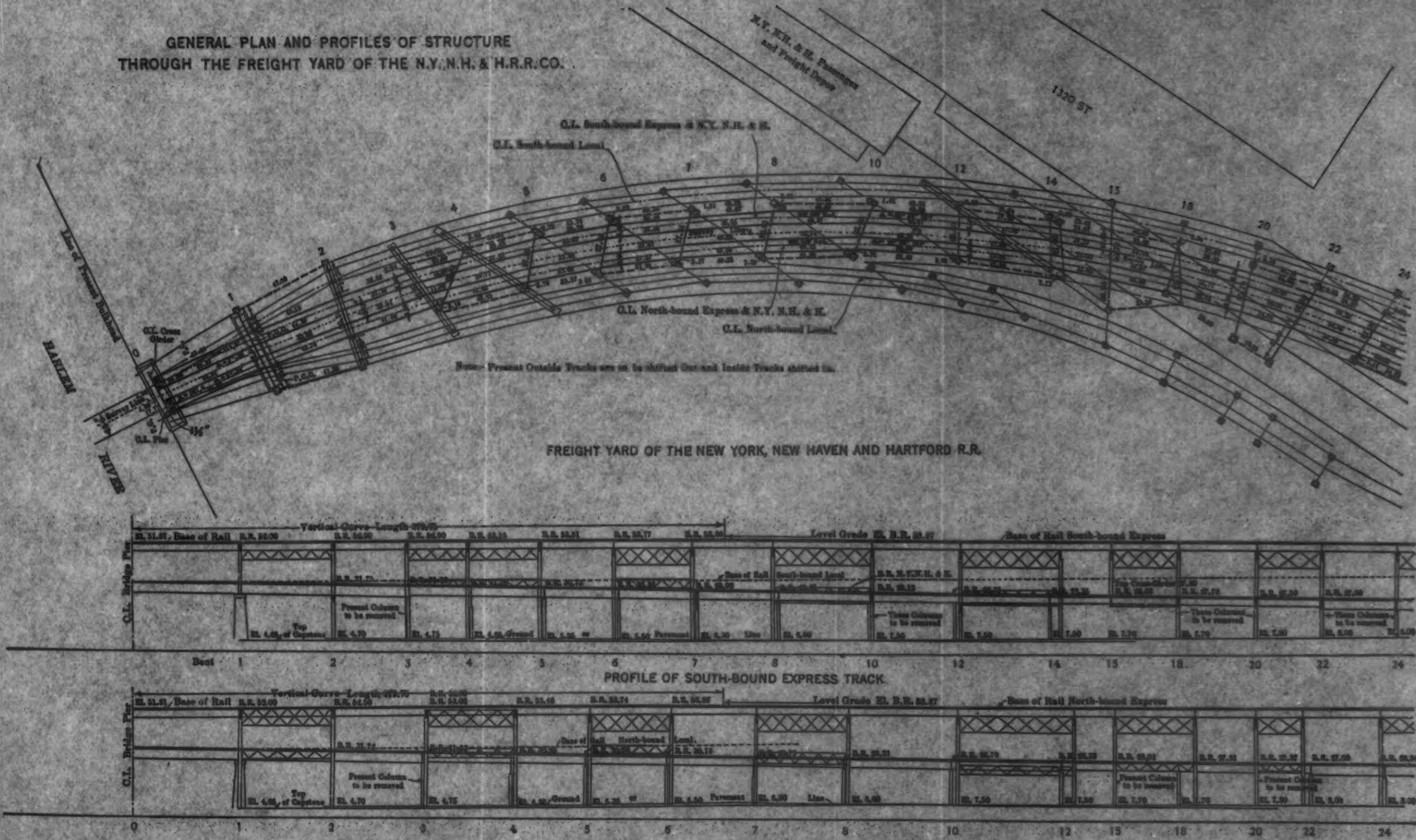
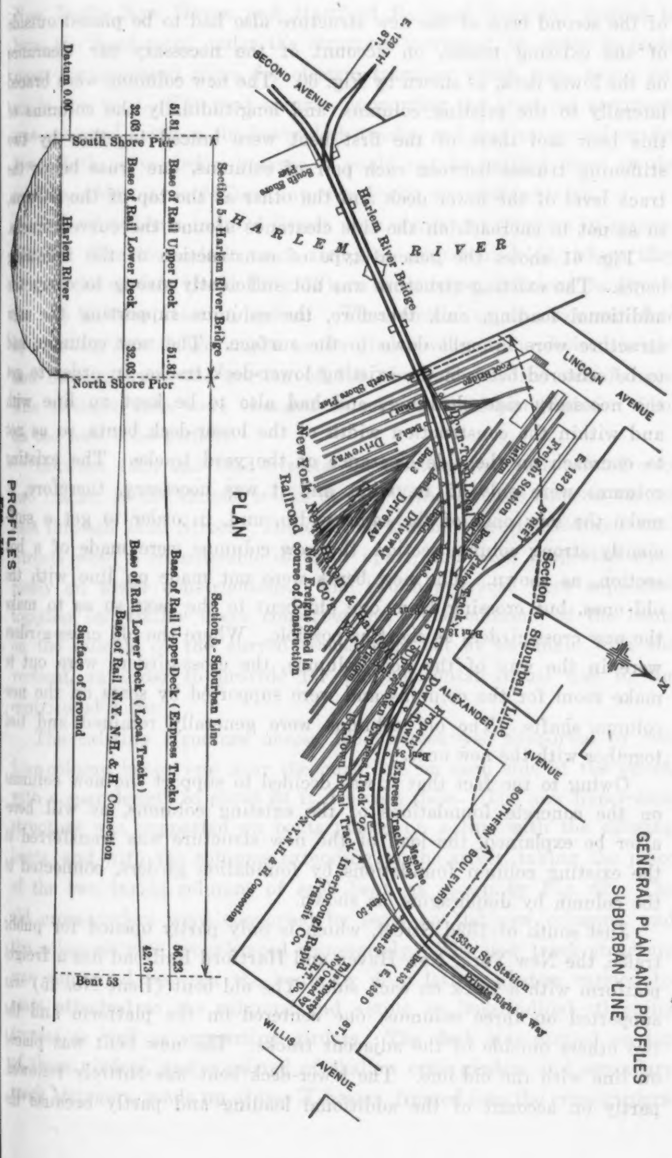
GARDINER AND JOHANNESSON ON
ELEVATED RAILWAY IMPROVEMENTS.

GENERAL PLAN AND PROFILES OF STRUCTURE
THROUGH THE FREIGHT YARD OF THE N.Y.N.H. & H.R.R.CO.



PROFILE OF NORTH-BOUND EXPRESS TRACK

PROFILES



GENERAL PLAN AND PROFILES
SUBURBAN LINE

of the second bent of the new structure also had to be placed outside of the existing tracks, on account of the necessary car clearance on the lower deck, as shown by Fig. 60. The new columns were braced laterally to the existing columns, and longitudinally the columns of this bent and those of the first bent were braced together by two stiffening trusses between each pair of columns, one truss below the track level of the lower deck and the other at the top of the column, so as not to encroach on the side clearance around the curved tracks.

Fig. 61 shows the general type of construction of the following bents. The existing structure was not sufficiently strong to carry the additional loading, and, therefore, the columns supporting the new structure were brought down to the surface. The new columns had to be centered between the existing lower-deck tracks, in order to get the necessary side clearance, and had also to be kept on line with and within the construction width of the lower-deck bents, so as not to encroach on the side clearance of the yard tracks. The existing columns were only 12 in. wide, and it was necessary, therefore, to make the new ones of the same width, and, in order to get a sufficiently strong column section, the new columns were made of a box section, as shown. The new bents were not made on line with the old ones, but crossing from one old bent to the next so as to make the new cross-girders as short as possible. Where the old cross-girders were in the way of the new columns, the cross-girders were cut to make room for the columns and were supported by seats on the new column shafts. The old columns were generally retained and tied together with the new ones.

Owing to the fact that it was decided to support the new columns on the concrete foundations of the existing columns, as will hereafter be explained, the load of the new structure was transferred to the existing column foundations by foundation girders, connected to the column by diaphragms, as shown.

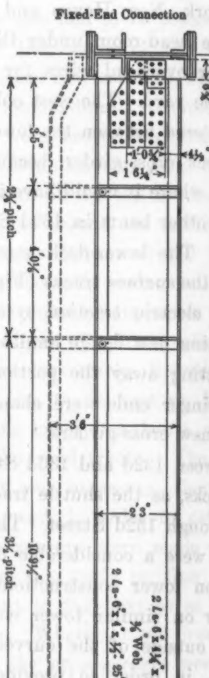
Just south of 132d Street, which is only partly opened for public traffic, the New York, New Haven and Hartford Railroad has a freight platform with a track on each side. The old bent (Bent No. 15) was supported on three columns, one centered on the platform and the two others outside of the adjacent tracks. The new bent was placed on line with the old one. The lower-deck bent was entirely renewed, partly on account of the additional loading and partly because the

New York, New Haven and Hartford Railroad Company desired to have the head-room under the structure increased, in order to make room for overhead wires for electric traction, which were being put up in the yard. The west column, supporting the overhead structure, was centered between the lower-deck tracks, and was supported on the lower-deck cross-girder, because it could not be carried down to the surface, where it would have interfered with the surface track.

The other bents in 132d Street were similar to this one in general details. The lower-deck track stringers were remodeled, where they crossed the surface tracks (Fig. 73), in order to get sufficient head-room for the electric traction system. The remodeling was accomplished by riveting new flange angles to the girder webs at the desired height and cutting away the portion of the girders below the new flanges. The stringer ends were changed to rest on seats provided for them on the new cross-girders.

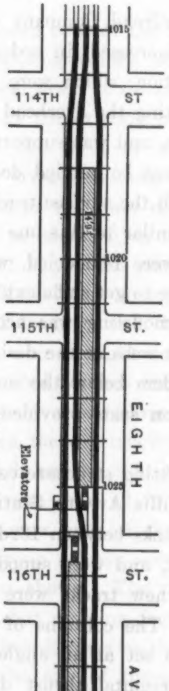
Between 132d and 133d Streets, the existing structure carried only two tracks, as the shuttle tracks at the Willis Avenue Station turned east through 132d Street. The existing tracks between 132d and 133d Streets were a considerable distance apart, and were supported separately on tower constructions. The two new tracks were supported together on similar tower constructions. The columns of the bents on the outside of the curved tracks were set at an angle with the vertical, in order to provide for the horizontal thrust due to the centrifugal force.

The existing structure across 133d Street was supported by two four-column bents, one near the curb line on each side of the street, with a span length of about 64 ft. between them. The new upper-deck structure was supported on bents placed on a line with the existing bents, and with the columns extending to the street, taking the place of the two inside columns of each bent, as shown by Fig. 62. The old cross-girders were supported by seats on the new columns, and the columns were cross-braced transversely. The new track structure was supported between the bents by deep lattice girders carried by seats attached to the columns and acting as longitudinal stiffening trusses as well as supporting girders. The deck was carried on top of these girders, and consisted of shallow cross-girders and secondary track stringers, made up of two I-beams, framed into the cross-girders.



REMODELING OF GIRDERS IN N.Y.N.H. & H.R.R. YARD

FIG. 73.



116TH STREET STATION, NINTH AVE. LINE

FIG. 74.

Details of Design.—Section No. 6-A.—Section No. 6-A extended from 133d to 147th Street. The portion from 133d to 145th Street is on a private right of way, 50 ft. wide; the remainder is in Third Avenue. The existing structure was a two-track line, and, through the right of way, consisted of track stringers supported on masonry piers, except at street crossings, where they were supported on column bents. There were stations with center platforms at 133d, 138th, and 143d Streets. The new work included the addition of two overhead tracks from 133d Street to a point north of 143d Street, where the two tracks joined and ran as a single track down to the grade of the lower-deck track at 145th Street, and continued on that grade as an additional center track. The new track layout also included a single-track ramp running from the upper deck at 138th Street to the lower deck at 141st Street, connecting at each end with both main tracks. At 143d Street provision was made for turn-outs of both the new upper-deck tracks to connect with a new extension between this line and the existing elevated portion of the subway at Brook and Westchester Avenues. New overhead island platforms were added to the stations at 133d, 138th, and 143d Streets, and the existing island platforms on the lower deck were entirely rebuilt. Prior to the erection of the new structure, the existing south-bound track was moved away from the north-bound track to make room for the new structure, as will be described later.

Fig. 63 shows a cross-section of the structure at the 133d Street Station. The new upper-deck structure, as well as the widened lower-deck platform, was carried on new column bents, about 40 ft. from center to center. The lower-deck platforms were supported on two deep stringers carried by seats on the columns, as well as by a center stringer framed into and supported by cross-frames between the columns and between the centers of the deep stringers, so that the span of this center stringer was only about 20 ft. The upper-deck structure had both platforms and tracks supported on columns spliced to the lower-deck columns, and also by intermediate columns, supported on top of the deep platform stringer, making the spans of the upper-deck track structure only about 20 ft., which was desired in order to make the construction height and the distance between the two platform levels as small as possible. Fig. 64 shows the details of the lower-deck bent. The columns were of the standard cross-section, 15 in. square, and braced transversely. The channels of the column

shafts were turned with their webs parallel to the bent, and the columns, therefore, were stiff enough to resist longitudinal stresses without additional bracing. The column bases were riveted to the columns, and designed to resist the bending stresses; they were fastened to the foundations by four anchor-bolts.

The construction at the 138th Street Station is similar to that at the 133d Street Station. Between these stations, the new structure consisted of two sets of track stringers supported on column bents which, in every third span, were braced longitudinally to form a tower to resist the longitudinal forces; the bents between the towers were designed for vertical and transverse loads only.

Fig. 65 shows the new structure at the crossing of 135th Street. The supporting columns were centered under the tracks, and were braced both horizontally and transversely from the top to a point about 14 ft. above the street level. The cross-girder was supported on the top of the column, and its center portion, which did not carry any load, but acted as transverse bracing, was designed with a lattice web; the end portions, which carried the load, had solid webs.

North of 138th Street, the center ramp, leading from the upper to the lower deck, commenced. Fig. 66 shows the supporting cross-bent near the top of the ramp. The details of the bent are similar to those shown by Fig. 65, except that the cross-girder is a plate girder throughout. The outside track stringers are carried on the top of the cross-girder, and the center-track stringers by seats attached to the web of the cross-girder. Fig. 67 shows the cross-bent at an intermediate point of the ramp. On account of the necessary clearance for cars on the center track, the column tops could not be braced transversely. The column section, therefore, was strengthened by increasing it in width and making the flanges of four angles and a cover-plate. The web-plate stopped about 4 ft. 6 in. from the top of the column, and, in its place, was inserted a twin bracket the web of which supported the track stringers. When the center track was sufficiently low, the column tops were braced transversely by an arched plate girder. Between the 133d and 138th Street Stations some of the column bents were also braced longitudinally to form towers.

The same construction was used between 144th and 145th Streets, where the new structure carried only a single track. From 145th Street to the end of the section at 147th Street, the new work consisted

simply of adding center-track stringers to the existing structure, and replacing the old cast-iron column bases with new cast-steel bases.

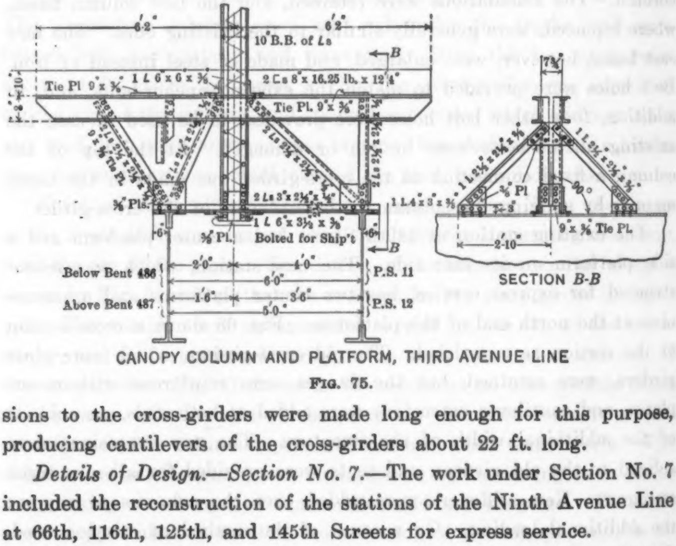
Details of Design.—Section No. 6-C.—Through the greater portion of this section, which extended, on Third Avenue, from 147th Street to Fordham Road, the existing columns were replaced. These columns were only 12 in. square, and consisted of two 12-in. channels latticed together. The new columns were built up of two 15-in. channels, four angles, and a web-plate, but were only made 12 in. wide, as the franchise rights of the line did not permit wider columns. The existing columns were bolted to shallow cast-iron bases, which were fastened to the granite caps of the foundations, the latter being 18 in. thick. The only weak member for the added loading was the column. The foundations were retained, and the new column bases, where replaced, were generally similar to the existing ones. The new cast bases, however, were enlarged, and made of steel instead of iron. Bolt holes were provided to match the existing anchor-bolts, and, in addition, four other bolt holes were provided, to be used in case the existing anchor-bolts were broken or damaged. At the top of the column a fixed connection to the cross-girder was made in the usual manner by splicing the column to the stiffeners of the cross-girder.

The existing station at 149th Street had a center platform and a side platform on its east side. The new station, which was reconstructed for express service, has two center platforms and a mezzanine at the north end of the platforms. Fig. 68 shows a cross-section of the station as remodeled. The old cross-girders, which were plate girders, were retained, but the flanges were reinforced with cover-plates, and cantilever extensions were added at both ends, on account of the additional width of the structure. The new extensions were spliced to the old girders, and seats were provided for all new track stringers. New stiffeners were added over the columns to support the additional loading. On account of the method of erection used, the old track stringers were retained in place and used for the support of the new platforms. The new platform structure is shown by Fig. 75. The platform beams were made of two channels each, and were supported on frames of light angles and stiffened against overturning by angle braces to the supporting stringers.

The stations at Tremont Avenue and Fordham Road were also reconstructed for express service with two island platforms. The

general details of the steelwork were similar to those of the 149th Street Station.

The existing stations at 156th, 161st, 166th, 169th Streets, Claremont Parkway, 174th, and 183d Streets, had center platforms. All these stations were reconstructed with side platforms, so as to make room for the express track. Fig. 69 shows a typical cross-section of the remodeled structure, and Fig. 76 shows the details of the extensions added to the existing cross-girders in order to provide supports for the side platforms. The station building was generally placed on a mezzanine, and was connected with the platforms by stairs at the outside edge of the platforms. As it was desired to support the added structure without the use of additional columns, the exten-



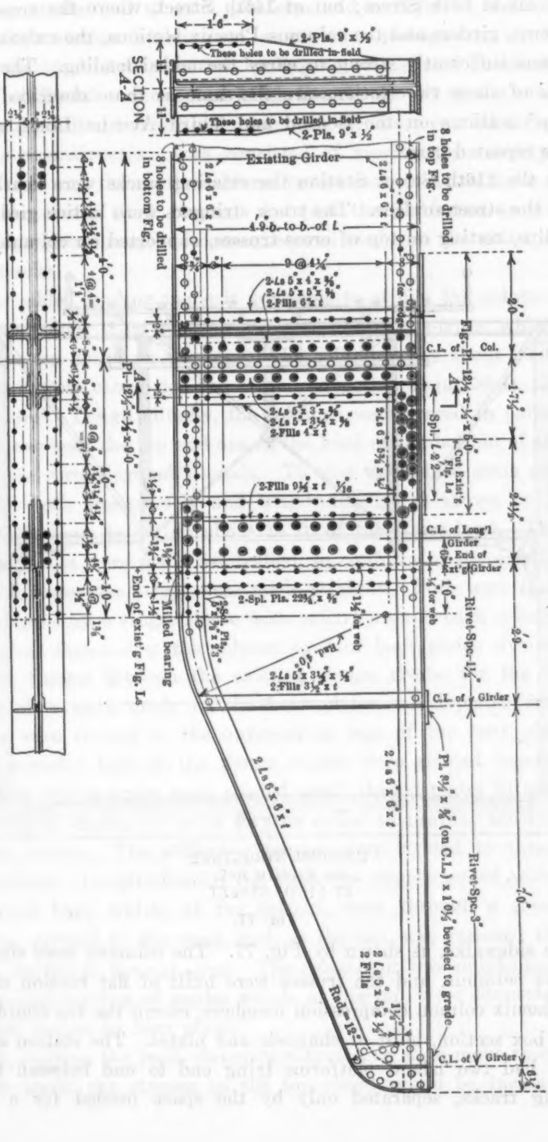
CANOPY COLUMN AND PLATFORM, THIRD AVENUE LINE

FIG. 75.

sions to the cross-girders were made long enough for this purpose, producing cantilevers of the cross-girders about 22 ft. long.

Details of Design.—Section No. 7.—The work under Section No. 7 included the reconstruction of the stations of the Ninth Avenue Line at 66th, 116th, 125th, and 145th Streets for express service.

The existing structure at 66th Street was similar to that in Third Avenue, and carried three tracks; the structure at 145th Street was similar to that of Second Avenue, and carried also three tracks before the reconstruction. At both these stations the structure was low, and, therefore, the express stations were made of the "hump" type. On account of the added loading, the lower-deck cross-girders and columns were replaced with new material throughout the length of the new

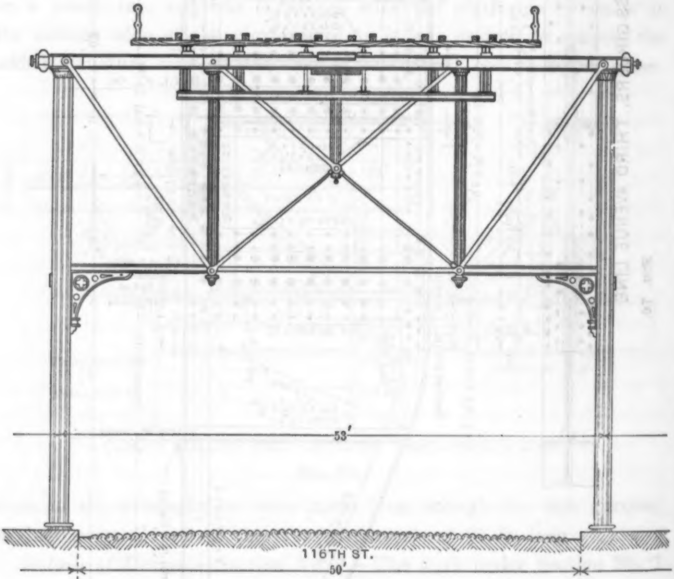


CROSS-GIRDERS, THIRD AVENUE LINE

FIG. 76.

platforms at 66th Street; but at 145th Street, where the cross-girders were twin girders and the columns Phœnix sections, the existing structure was sufficiently strong to carry the added loading. The general details of these two stations are the same as those described for the "hump" stations on the Second and Third Avenue Lines, and will not be repeated here.

At the 116th Street Station the existing tracks were nearly 50 ft. above the street surface. The track stringers were lattice girders with 7-in. lips, resting on top of cross-trusses, supported on columns placed



ORIGINAL STRUCTURE
ON EIGHTH AVE.
AT 116TH STREET.

FIG. 77.

on the sidewalks, as shown by Fig. 77. The columns were six-section Phœnix columns, and the trusses were built of flat tension members and Phœnix column compression members, except the top chord, which was a box section, built of channels and plates. The station at 116th Street had two island platforms lying end to end between the two existing tracks, separated only by the space needed for a station

building at the end of each platform and a flight of stairs leading from each station building to a common passageway immediately below the track structure and leading to an elevator building at the corner of Eighth Avenue and 116th Street. South of the south platform and north of the north platform, the structure carried three tracks.

The reconstruction work consisted of moving the south platform to the west and the north platform to the east, so as to make room to carry the center track through and past the platforms, as shown by Fig. 74, which change, of course, necessitated also the shifting of the local tracks.

The added loading made it advisable to stiffen the column shafts, so as to reduce their bending stresses. The cross-truss, which rested with the top chord on the top of the columns, was about 20 ft. deep, and its bottom chord tied the columns together about 30 ft. above the street level. Longitudinally, the columns were braced in pairs by two struts, one near the top and one at the level of the bottom of the cross-truss, and two diagonal tie-rods. To this was added some additional bracing, both transversely and longitudinally, as shown by Fig. 70, extending down to within about 14 ft. of the street level. The longitudinal struts were made fish-bellied, to conform to the existing ones, and the transverse struts were made straight. Both were fastened to the columns by a single sleeve with collars around each column. The sleeve was fastened to the column by four bent plates riveted to the column flanges between the existing flange rivets, for the heads of which slots were made in the bent plates. Web-plates and flange angles were riveted to the outstanding legs of the bent plates, and the horizontal legs of the flange angles were riveted together with plates, to which again were riveted small shaped plates fitting around the column shafts, so as to form a collar to prevent tension in the column rivets. The stiffening trusses were riveted to these sleeves and collars. Longitudinally, the strut was supplemented with a cross of round bars, which, at the bottom, went through a special steel casting, riveted to the strut, and, at the top, was fastened through a yoke, riveted to the old strut. The cross-strut was supplemented with a diagonal bracing of angles which, at the top, were connected to the column flanges by bent plates.

By shifting the track stringers sidewise, so as to make room for the center track, the stresses in the top chord would be increased, and,

therefore, it was decided to reinforce it. As it was not desired to do the reinforcing in such a manner that shoring should be needed during the work, the method shown by Fig. 70 was adopted. The portion to be reinforced was that between the column and the first vertical post. A lattice box-girder of the same width as the top chord was suspended from it by two yokes, one at each end. Plates were then driven in between the reinforcing girder and the top chord, and secured with rivets.

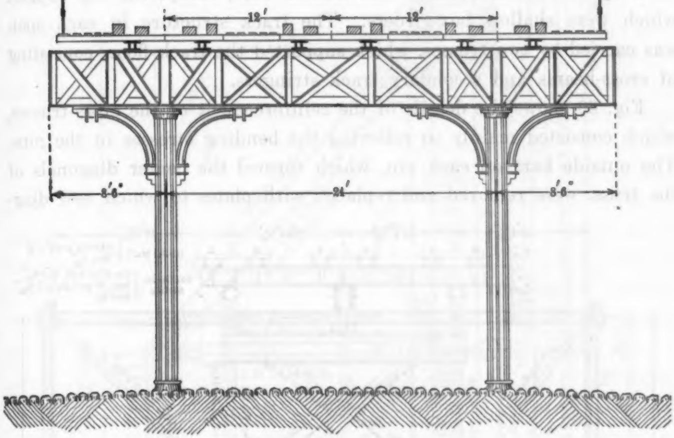
In addition to the reinforcing of the columns as described, it was originally contemplated to reinforce the connection of the columns to the foundations, but, as this was found to be unnecessary, it was not carried out.

A new mezzanine station was constructed under the structure at the level of the existing footbridge leading to the elevator building. This new station was carried by two new twin girders, connected to the columns of adjacent bents in a manner similar to that previously described for connecting the transverse and longitudinal struts.

The existing structure at the 125th Street Station was similar to that on the Second Avenue Line, and is shown in cross-section by Fig. 78. The same arrangement of platforms existed here as at the 116th Street Station, and the new arrangement was similar to that described for that station, except that the station buildings were retained. As the existing cross-girders were not long enough to carry the full width of the new structure, short extensions were added to them at each end, with seats to carry the outside stringers supporting the local tracks in their new position.

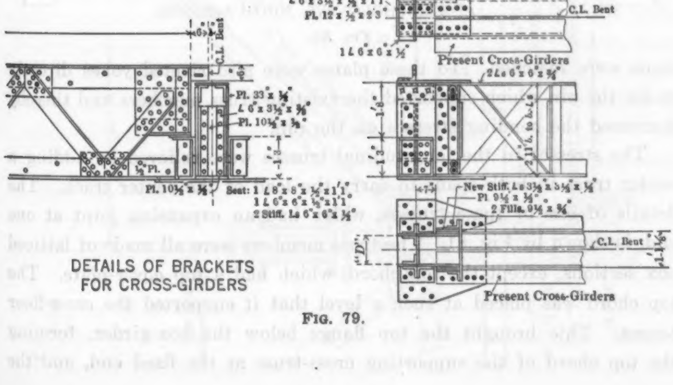
Fig. 79 shows the details of the cross-girder extensions and the new track stringers. Two vertical plates were riveted to the flange angles of the twin girder, and extended beyond it the full width of the new seats. Two vertical diaphragms stiffened these plates, one at the cross-girder end and the other at the center of the seats. Angles were riveted to the top and the bottom of the vertical plates, and were connected to the top and bottom flanges of the cross-girder by horizontal splice-plates. Regular seats were provided on the vertical plates for the new track stringers. The track stringers were lattice girders, in order to conform to the existing stringers.

South of the 125th Street Station, where St. Nicholas Avenue crosses the elevated structure, were two spans, one of about 99 ft.



ORIGINAL STRUCTURE
ON EIGHTH AVE.
AT 125TH STREET.

FIG. 78.

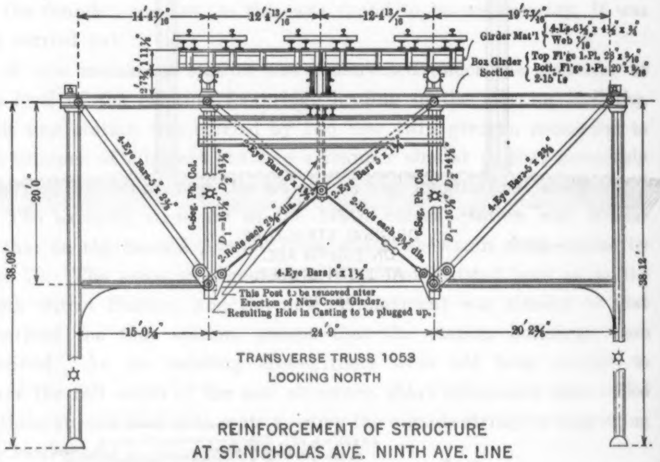


DETAILS OF BRACKETS
FOR CROSS-GIRDERS

FIG. 79.

and the other about 80 ft. long, which required reinforcing, in order to carry the load of the three tracks. The existing cross-bents were built up of Phœnix sections and tension bars, except the top flanges, which were shallow box-girders. The track structure in each span was carried by two trusses, which supported the track floor, consisting of cross-beams and secondary track stringers.

Fig. 80 shows the details of the reinforcement of the cross-trusses, which consisted mainly in relieving the bending stresses in the pins. The outside bars on each pin, which formed the center diagonals of the truss, were removed and replaced with plates to which new diag-

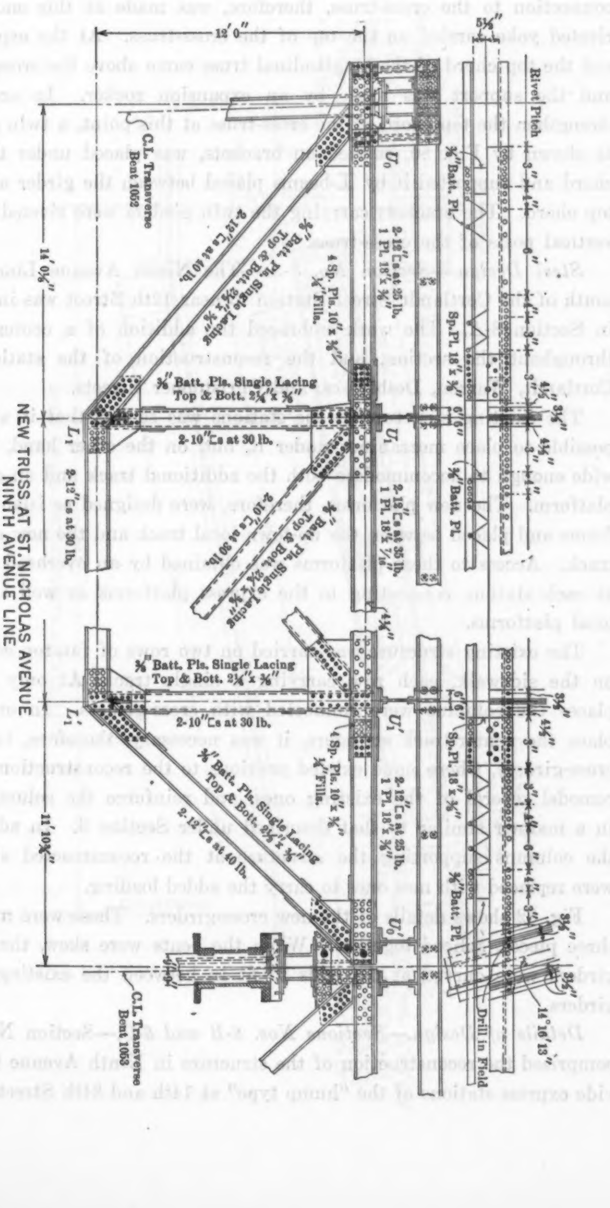


REINFORCEMENT OF STRUCTURE
AT ST. NICHOLAS AVE. NINTH AVE. LINE

FIG. 80.

onals were fastened. To these plates were also riveted yokes directly under the pin which supported the existing truss members and thereby decreased the bending stresses on the pin.

The stresses in the longitudinal trusses were relieved by adding a center truss in both spans to carry the load of the center track. The details of one of these trusses, which had an expansion joint at one end, is shown by Fig. 81. The truss members were all made of latticed box sections, except the top chord, which had a top cover-plate. The top chord was placed at such a level that it supported the cross-floor beams. This brought the top flange below the box-girder, forming the top chord of the supporting cross-truss at the fixed end, and the



NEW TRUSS AT ST. NICHOLAS AVENUE
NINTH AVENUE LINE

Fig. 81.

connection to the cross-truss, therefore, was made at this end by a riveted yoke carried on the top of the cross-truss. At the expansion end the top chord of the longitudinal truss came above the cross-truss, and the support was made by an expansion rocker. In order to strengthen the top chord of the cross-truss at this point, a twin girder, as shown by Fig. 80, carried on brackets, was placed under the top chord and supported it by I-beams placed between the girder and the top chord. The brackets carrying the twin girders were riveted to the vertical posts of the cross-truss.

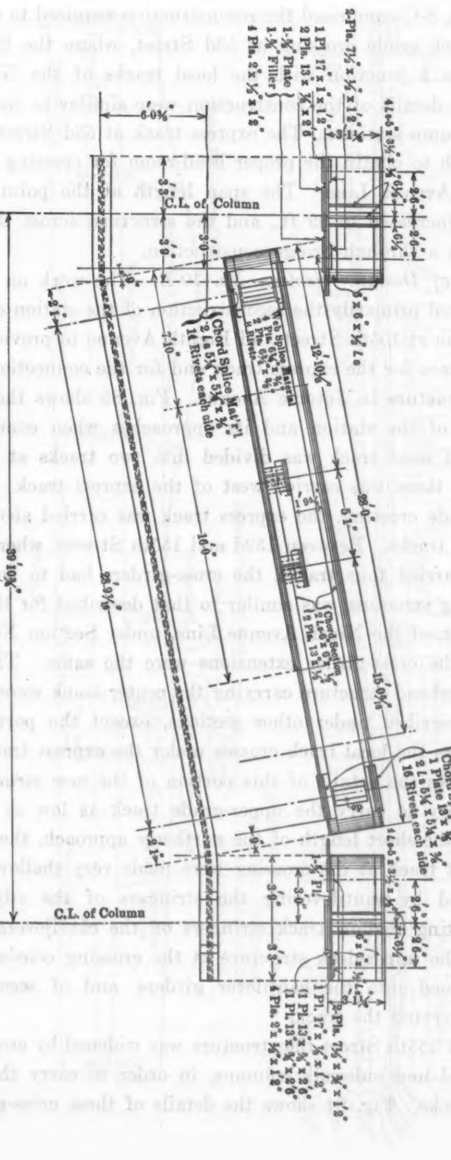
Steel Design.—Section No. 8-A.—The Ninth Avenue Line from south of the Cortlandt Street Station to near 12th Street was included in Section 8-A. The work embraced the addition of a center track throughout the section, and the reconstruction of the stations at Cortlandt, Warren, Desbrosses, and Christopher Streets.

The existing structure at the stations was so low that it was not possible to place mezzanines under it, but, on the other hand, it was wide enough to accommodate both the additional track and an express platform. The new platforms, therefore, were designed as island platforms and placed between the up-town local track and the new express track. Access to these platforms was obtained by an overhead bridge at each station, connecting to the express platforms as well as both local platforms.

The existing structure was carried on two rows of fan-top columns on the sidewalk, each row carrying a single track. At only a few places the columns were connected with cross-girders. In order to place the center-track stringers, it was necessary, therefore, to place cross-girders, where none existed previous to the reconstruction work, remodel several of the existing ones, and reinforce the column tops in a manner similar to that described under Section 3. In addition, the columns supporting the structure at the reconstructed stations were replaced with new ones to carry the added loading.

Fig. 82 shows details of the new cross-girders. These were made in three pieces, spliced together. When the bents were skew, the cross-girders were curved at the ends to fit in between the existing track girders.

Details of Design.—Sections Nos. 8-B and 8-C.—Section No. 8-B comprised the reconstruction of the structure in Ninth Avenue to provide express stations of the "hump type" at 14th and 34th Streets; and



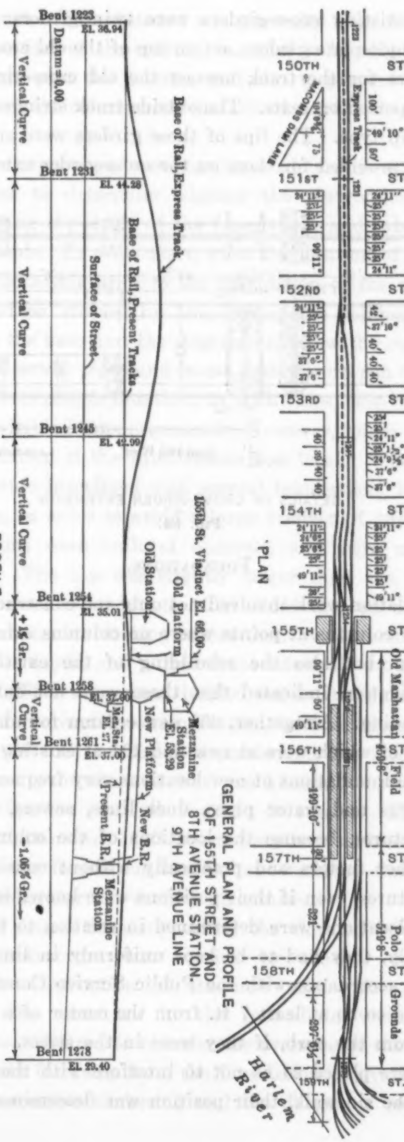
SKEW CROSS-GIRDER. NINTH AVENUE LINE

FIG. 82.

Section No. 8-C comprised the reconstruction required to eliminate the express track grade crossing at 53d Street, where the Sixth Avenue Line makes a junction with the local tracks of the Ninth Avenue Line. The details of the construction were similar to those described for other hump stations. The express track at 53d Street was carried high enough to obtain the proper head-room for crossing the track of the Sixth Avenue Line. The span length at the point of crossing had to be increased to 99 ft., and the structure across this span was designed as a through bridge construction.

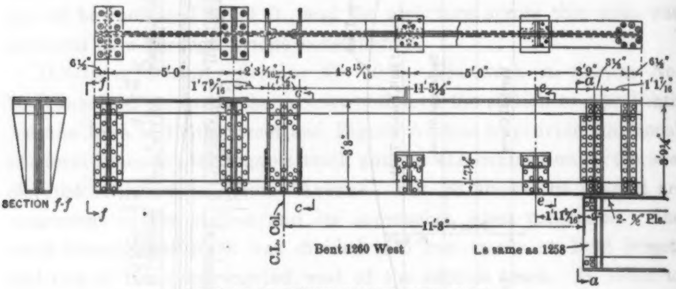
Details of Design.—Section No. 10-B.—The work on Section No. 10-B involved primarily the reconstruction of the station on the Ninth Avenue Line at 155th Street and Eighth Avenue to provide additional platform space for the express track and for the connection to the new elevated structure in Jerome Avenue. Fig. 83 shows the general arrangement of the station and its approaches when completed. The north-bound local track was divided into two tracks at 152d Street, and one of these was carried west of the express track. In order to avoid a grade crossing, the express track was carried above the grade of the local tracks. Between 152d and 155th Streets, where the rebuilt structure carried four tracks, the cross-girders had to be lengthened. The existing structure was similar to that described for the station at 125th Street, of the Ninth Avenue Line, under Section No. 7, and the details of the cross-girder extensions were the same. The details of the new overhead structure carrying the center track were also similar to those described under other sections, except the portion at 152d Street, where the local track crosses under the express track structure. Fig. 71 shows the details of this portion of the new structure. As it was necessary to carry the upper-grade track as low as possible, on account of the short length of the northerly approach, the track stringers of that track at the crossing were made very shallow. This was accomplished by cantilevering the stringers of the adjacent spans and supporting shallow track stringers on the cantilevers. The floor system of the upper-deck structure at the crossing consisted of floor-beams, framed into the cantilever girders, and of secondary track stringers carrying the track.

North of 155th Street the structure was widened by cross-girder extensions and new sidewalk columns, in order to carry the additional westerly tracks. Fig. 84 shows the details of these cross-girder exten-



PROFILE
FIG. 83.

sions. The existing cross-girders were twin girders; the extensions were made single-plate girders, set on top of the old cross-girders. The track stringers for the track nearest the old cross-girders were new, and were supported on seats. The outside track stringers were old, and of the 7-in. lip type. The lips of these girders were supported on top of wide seats provided for them on the cross-girder extensions.



DETAILS OF CROSS-GIRDER EXTENSION

FIG. 84.

FOUNDATIONS.

The foundation work involved not only the construction of foundations for new columns at points where no columns existed prior to the reconstruction, but also the rebuilding of the existing foundations where investigation indicated that these were not sufficient to carry the added loading. Altogether, 638 new column foundations were constructed, 444 of which were at new and 194 at existing locations. The placing of the foundations at new locations very frequently necessitated changes of gas and water pipes, duct lines, sewers, and other subsurface structures, because the locations of the columns were determined by other factors and practically without considering the subsurface structures, even if their positions were known in advance. The new column locations were determined in relation to the street traffic. Longitudinally, they had to be kept uniformly in line whenever possible and, in accordance with the Public Service Commission's certificate, they had to be at least 7 ft. from the center of a street-car track and 14 ft. from the curb, if they were in the street. At street crossings, they were placed so as not to interfere with the footwalk crossings; if on the sidewalk, their position was determined a definite dis-

tance from the curb. This distance was generally 8 in. from the edge of the curb to the face of the column, in order to be able to carry the curb stone through past the column, and also to prevent interference between the face of the column and the hub of a truck driving close to the curb.

If sub-surface structures were encountered, an investigation was always made first to determine whether the obstructions could be avoided by changing the shape of the foundation, and, where possible, the change was made. In some cases, when the location of the obstruction was known definitely prior to the fabrication of the columns, the interference could be avoided by lengthening the column shaft, and thereby bringing the base and the foundation below the obstruction.

At times small sewer pipes and house drains were run through and embedded in the concrete foundation; in such cases, cast-iron pipe was substituted for vitrified pipe; generally, however, pipes were offset around the foundation if the interference was local. When the same sub-surface structure interfered with several foundations, it was relaid in a new position, in order to avoid a large number of offsets.

The foundations were built of concrete, and were usually rectangular in plan. The top was slightly larger than the base of the column resting on the foundation. The sides tapered downward to the top of a rectangular footing which had vertical sides varying from 1 to 2 ft. in depth, and a horizontal offset all around, varying from 6 to 12 in. This shape was decided on, as it requires the least excavation and can be cast in one operation, without changing forms. The bottom area of the foundation depended on the load on it and the carrying capacity of the soil, but was generally not less than 7 ft. square. Usually, the size was 9 or 10 ft. square. The standard depth of the foundations was 14 ft. below the street surface; this depth generally assured the foundation against undermining due to excavations for future cellars or sub-surface structures.

Where the old foundations supporting the elevated structure were rebuilt, the superstructure was carried by shoring during their removal and the construction of the new ones. As the shoring had to support the structure in the immediate vicinity of the column, while it was desired to transfer the pressure at the street level to points sufficiently far away from the foundation excavation to avoid the necessity of excessive timbering and the danger of settlement of

the shoring due to a cave-in, the load was transferred by the shoring to girders supported at the ends by blocking, laid on the street surface. The girders were long enough to insure that the load on them would not produce excessive horizontal pressures on the sheathing holding the sides of the excavation. In most cases the shoring girders were either 24-in., 120-lb., Bethlehem beams, or 48-in. plate girders from 40 to 45 ft. long. The 24-in. beams were preferred, when strong enough to carry the load, as they were more easily handled. The blocking at the ends consisted usually of old ties laid side by side and wedged up slightly above the street surface, which was not disturbed. After the blocking and the girders were set in place and leveled up, the space between the blocking and the street surface was filled with a lean liquid mortar of cement and sand. It was found that this mortar when set produced an excellent bearing, and could easily be removed from the street surface, when the work was completed. No settlement of the street surface was ever observed due to the pressure of the shoring.

When conditions permitted, the shoring consisted simply of the shoring girders and brackets riveted to the columns and supported by the girders. This method was used when the structure had sufficient transverse stiffness, but only if the columns were to be renewed, as it was not desired to drill the holes necessary for the connection of the brackets in the webs or flanges of permanent columns. The column load was taken off the old foundations and transferred to the shoring girders by driving wedges between the brackets and the girders. When these wedges were being driven, the first effect was to deflect the girders, without raising the column; as the driving continued, the column, which had previously been made free to move vertically, by loosening the nuts of the anchor-bolts, would be lifted clear of the foundation. The driving of the wedges would be continued until the column was raised about ½ in. above the foundation. No work of removal of the foundation would be commenced until the column had remained wedged up for about 24 hours, in order to test the shoring under traffic. The wedging operation was carried on between trains, in order to avoid the additional live load due to the trains passing over the columns.

In most cases the structure of the single-column type was shored by placing four-post towers, made up of 12 by 12-in. yellow pine timbers on top of the shoring girders, one tower at each side of the

column. The structure was raised off the old foundation by wedging between the top of the towers and the bottom of the track stringers, the column which was connected to the track stringers being raised with the stringers. Special types of shoring used under extraordinary conditions will be described later.

The sides of the excavations were supported by 2-in. close sheathing, held in place by rangers, which were generally made of old 6 by 8-in. track timber, scarfed at the ends. The sheathing was driven down as the excavation proceeded.

At first the old foundations to be replaced were removed by hand. This method, however, was slow and expensive, and, as soon as air supply could be had, pneumatic hand drills were used instead.

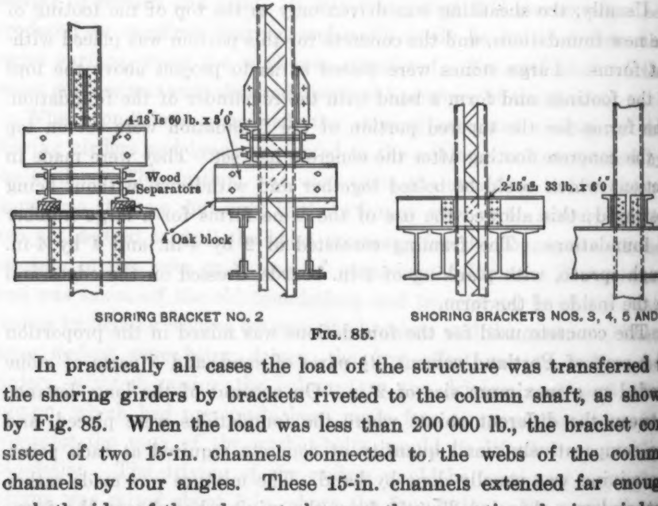
Usually, the sheathing was driven only to the top of the footing of the new foundations, and the concrete for this portion was placed without forms. Large stones were placed so as to project above the tops of the footings and form a bond with the remainder of the foundation. The forms for the tapered portion of the foundation were set on top of the concrete footing after the concrete had set. They were made in sections which could be bolted together and withdrawn without being destroyed; this allowed the use of the same forms for a large number of foundations. The framing consisted of 2 by 4-in. and 4 by 4-in. rough spruce, with planking of 2-in. boards, dressed on the edges and on the inside of the form.

The concrete used for the foundations was mixed in the proportion of 1 part of Portland cement, 2½ parts of sand, and 5 parts of stone graded to a maximum size of 2 in. On account of the long distance between the different points where the foundations were placed, and the comparatively small quantity of concrete required at each point, the mixing was usually done by hand. The mixture was made rather wet, and was shoveled directly from the mixing board into the form. A laborer was generally stationed inside the form to distribute the concrete and work it well up against the sides.

The greater part of the foundation work occurred on Sections Nos. 1, 2-A, 2-B, 5-A, 5-B, 5-D, and 6-A, and only this work will be described herein.

Foundations.—Section No. 1.—Altogether, 64 foundations were constructed in this section, of which 52 replaced existing ones, and 12 were in new locations. In several places, where shoring was needed,

the gas mains were found to be so close to the street surface that provision had to be made against danger to the shoring from possible explosions and fire due to leaks in the mains, which danger often was aggravated by the necessity of placing the support of the shoring directly over the gas mains. This danger was provided against by drilling and plugging holes in the mains about 250 ft. apart. If a leak occurred, the flow of gas was stopped by bagging the adjacent holes until repairs were made, and if the leak was directly under the shoring, where no repairs could be made during the time the shoring carried the load, the pipe was by-passed around the shoring, and the interrupted house service connections were connected temporarily to the by-pass.



SHORING BRACKET NO. 2 SHORING BRACKETS NOS. 3, 4, 5 AND 6

FIG. 85.

In practically all cases the load of the structure was transferred to the shoring girders by brackets riveted to the column shaft, as shown by Fig. 85. When the load was less than 200 000 lb., the bracket consisted of two 15-in. channels connected to the webs of the column channels by four angles. These 15-in. channels extended far enough on both sides of the column to bear on the supporting shoring girders. When the load was greater, the brackets consisted of inverted seats riveted to the webs of the column shaft and supported on two 18-in. or 20-in. I-beams, which, again, were supported on I-beams laid crosswise so as to get a bearing on the shoring girders. Fig. 87 shows the structure in the New Bowery supported on brackets of the first type. Here the shoring girders of some of the bents were carried several feet above the sidewalk level, in order not to interfere with the surface structures, such as fire hydrants and coal holes.

Fig. 86.—New North Span of Harlem River Bridge.

Fig. 87.—Shoring for Foundations. New Bowery.

A special method of shoring was used for a column at Chatham Square. This column was so close to a surface-car track, that there was not sufficient room to place a shoring girder on the side of the column adjacent to the car track.

The shoring, as shown by Fig. 88, was accomplished by placing on the street surface two plate girders, 4 ft. deep, on the side of the

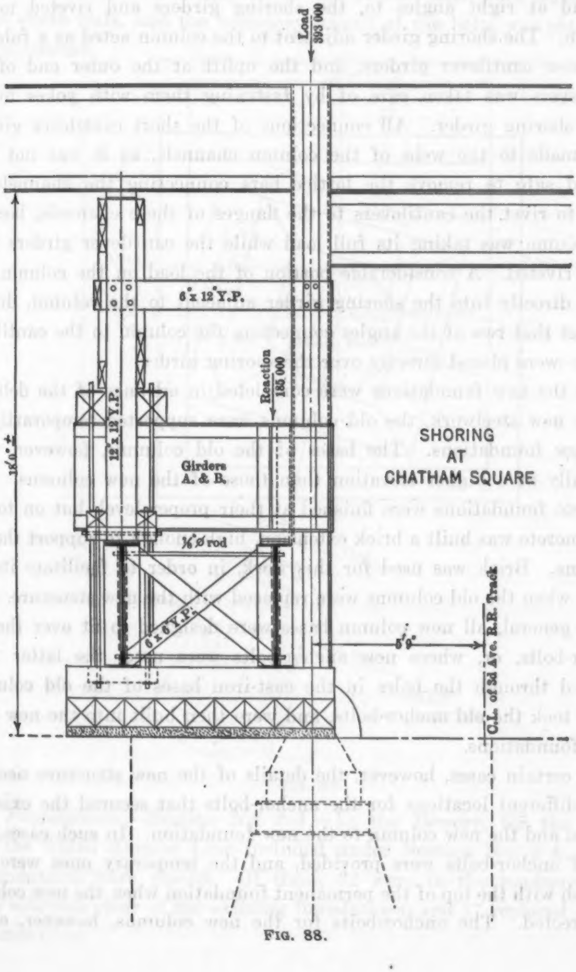


FIG. 88.

column away from the street-car tracks. The girders were placed approximately parallel to the car tracks, one close to the column and the other about 5 ft. from the first. By using timber bents, the load of two of the cross-girders carried by the column was brought to bear on the top of the second shoring girder. The remainder of the load on the column was carried by two short cantilever girders placed on top of, and at right angles to, the shoring girders and riveted to the column. The shoring girder adjacent to the column acted as a fulcrum for these cantilever girders, and the uplift at the outer end of the cantilevers was taken care of by fastening them with yokes to the outer shoring girder. All connections of the short cantilever girders were made to the webs of the column channels, as it was not considered safe to remove the lattice bars connecting the channels, in order to rivet the cantilevers to the flanges of these channels, because the column was taking its full load while the cantilever girders were being riveted. A considerable portion of the load on the column was taken directly into the shoring girder adjacent to the column, due to the fact that two of the angles connecting the column to the cantilever girders were placed directly over the shoring girder.

As the new foundations were completed in advance of the delivery of the new steelwork, the old columns were supported temporarily on the new foundations. The bases of the old columns, however, were generally at a higher elevation than those of the new columns. The concrete foundations were finished at their proper level, but on top of the concrete was built a brick extension, high enough to support the old columns. Brick was used for this work, in order to facilitate its removal when the old columns were replaced with the new structure.

In general, all new column bases were designed to fit over the old anchor-bolts, or, where new anchor-bolts were used, the latter were dropped through the holes in the cast-iron bases of the old columns which took the old anchor-bolts, and were then built into the new concrete foundations.

In certain cases, however, the details of the new structure necessitated different locations for the anchor-bolts that secured the existing column and the new column to the new foundation. In such cases, two sets of anchor-bolts were provided, and the temporary ones were cut off flush with the top of the permanent foundation when the new column was erected. The anchor-bolts for the new columns, however, could

not extend during the temporary period above the bottom of the old base. Fig. 89 shows such a case. As the new column was skewed in relation to the existing column, two sets of anchor-bolts were provided. The anchor-bolts for the new structure had sleeve nuts, set flush with the top of the permanent foundations. When the old steelwork was removed, extensions of the anchor-bolts were screwed into the sleeve nuts, and the necessary length of the bolts was obtained in this manner.

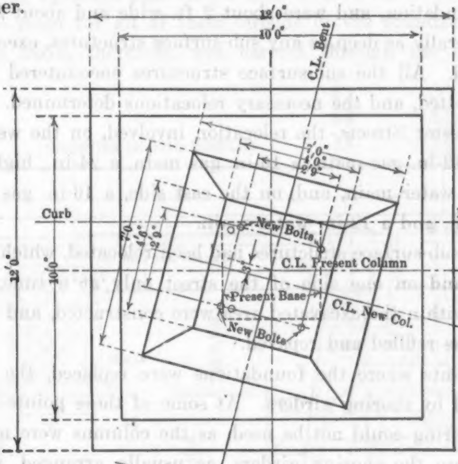


FIG. 89.

***Foundations.—Section No. 2-A.*—In the Bowery, for the portion of the Third Avenue Line included under Section No. 2-A, 66 new foundations were placed. Of these, 32 were in the roadway, 17 on the sidewalk clear of the existing foundations, and 17 replaced existing foundations.**

The Bowery is an old established main thoroughfare, and contains a vast number of sub-surface structures, the location of which it was desired to determine in advance, so that they could be removed where they interfered with the construction of the foundations. Trenches were excavated across the street, therefore, first on one side, between curb and car track, and then on the other side, so as to maintain the street traffic. These trenches were made on line with the location of each new foundation, and were about 2 ft. wide and about 5 ft. deep, which is generally as deep as any sub-surface structures, except sewers, will be found. All the sub-surface structures encountered were surveyed and plotted, and the necessary relocations determined. Between Canal and Hester Streets, the relocation involved, on the west side of the street, a 12-in. gas main, a 16-in. gas main, a 24-in., high-pressure (fire service) water main, and, on the east side, a 16-in. gas main, an 8-in. gas main, and a 12-in. water main.

After the sub-surface structures had been relocated, which was done in sections, and on one side of the street only at a time, the new foundations within the excavated area were constructed, and the whole excavation was refilled and repaved.

At the points where the foundations were replaced, the structure was supported by shoring girders. At some of these points the usual method of shoring could not be used, as the columns were near street crossings where the shoring girders, as usually arranged, would interfere with the traffic on the street-railway tracks. Therefore, the arrangement shown by Fig. 90 was used. An excavation, 10 by 8 ft., was made immediately adjacent to the foundation to be reconstructed, and carried down to the full depth of the new foundation, which was about 8 ft. below the existing one, and, therefore, necessitated careful sheathing. The bottom of the excavation was covered with concrete on which was laid a solid floor of 12 by 12-in. timbers. On the top of this floor was set a four-post timber bent with two of the posts vertical and the other two on a batter. The posts were 12 by 12-in. yellow pine timbers, and the bent was 19 ft. high. Near the column of the bent, next to that for which the foundation was to be reconstructed, another timber bent was placed, resting on the street and sidewalk surface. The shoring girders were placed on top of these bents and cantilevered 8½ ft. over the bent first described, so as to reach the

point of the structure to be supported. The uplift was taken care of by blocking the shoring girders against the track stringers.

Foundations.—Section No. 2-B.—In this section, which extended on the Bowery from Delancey to 5th Street, practically all the columns were moved from the curb line to the roadway. The greater part of the work consisted of changes in the sub-surface structures. On account of the excessive number of such structures, it was not possible to provide room for all of them outside of the foundations, and one 16-in. gas main, therefore, was carried through the center of the

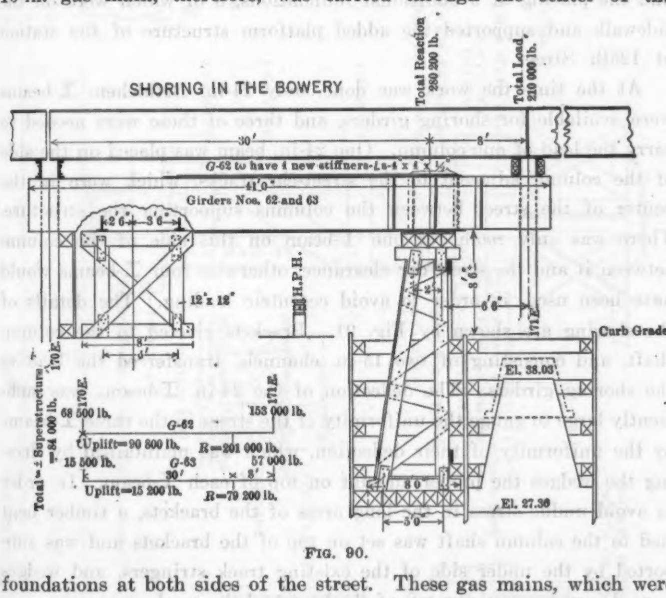


FIG. 90.

foundations at both sides of the street. These gas mains, which were of cast-iron pipe, were first jacked into such a position that they would be centered in the foundations throughout the whole length, and those portions of the mains which would be within the foundations were then replaced with wrought-iron pipe. In order to make it possible to withdraw and renew the pipe, a form was placed around it prior to placing the concrete. The form was made of a steel plate, semicircular at the top, and with straight sides. The bottom of the form was a 2-in. plank. The steel form was made in two lengths, meeting at the center of the pier, and was held in place by wooden

blocks, 1 in. thick, placed on top of the pipe. The form was removed by pushing these blocks beyond the form and allowing it to drop. The wrought-iron pipes were placed in such a manner that no joints occurred inside of the foundation, and the minimum height of the concrete above the pipe was 1 ft. The column bases supported by the foundations have been described previously.

Foundations.—Section No. 5-A.—The work in this section comprised the rebuilding of 26 foundations under the existing structure and the placing of 9 additional foundations, 8 of which were on the sidewalk and supported the added platform structure of the station at 125th Street.

At the time the work was done, only 24-in. Bethlehem I-beams were available for shoring girders, and three of these were needed to carry the load of one column. One 24-in. beam was placed on the side of the column adjacent to the street-car tracks, which were in the center of the street between the columns supporting the structure. There was only room for one I-beam on this side of the column between it and the street-car clearance, otherwise four I-beams would have been used, in order to avoid eccentric loading. The details of the shoring are shown by Fig. 91. Brackets riveted to the column shaft, and consisting of two 15-in. channels, transferred the load to the shoring girders. The deflection of the 24-in. I-beams was sufficiently large to gauge the uniformity of the stress in the three I-beams by the uniformity of their deflection, which was maintained by driving the wedges the proper amount on top of each I-beam. In order to avoid undue stress in the long arms of the brackets, a timber bent tied to the column shaft was set on top of the brackets and was supported by the under side of the existing track stringers, and wedges were driven between the top of the bent and the track stringers.

Foundations.—Section No. 5-B.—On this section no column foundations were removed and replaced, but 27 new foundations were built, of which 24 were built on the sidewalk to support the new 125th Street Station, and were of the standard type; the remaining three were of a special type, and deserve further description. These three foundations were in close proximity to the west shore line of the Harlem River, at the point where the bridge carrying the Second and Third Avenue Lines crosses the river. The surface of the ground is

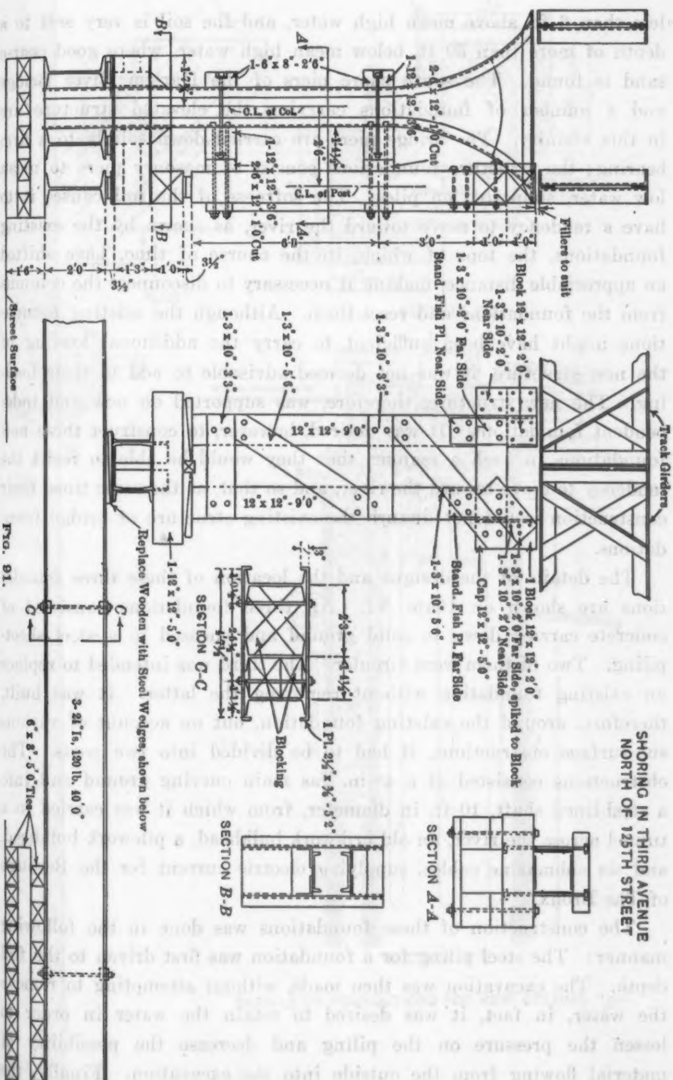

SHORING IN THIRD AVENUE
NORTH OF 125TH STREET
Track Girders
Fillers to suit
1 Block 12" x 12" - 2'0"
1-3" x 10" - 2'0" Near Side
1-3" x 10" - 2'0" Far Side
Stand. Fish Pl. Near Side
1-3" x 10" - 3'3"
1-3" x 10" - 5'6"
1-3" x 10" - 3'3"
12" x 12" - 9'0"
12" x 12" - 9'0"
1 Block 12" x 12" - 2'0"
1-3" x 10" - 2'0" Far Side, spiked to Block
1-3" x 10" - 2'0" Near Side
Cap 12" x 12" - 5'0"
Stand. Fish Pl. Far Side
1-3" x 10" - 5'6"
1-6" x 8" - 2'6"
1-12" x 12" - 3'6"
2-6" x 12" - 1'0" Cut
1-12" x 12" - 3'6"
2-6" x 12" - 1'0" Cut
C.L. of Col.
C.L. of Post
6'0"
3'0"
1'0"
1'0"
1'6"
2'0"
1'3"
1'0"
3½"
3½"
Street Surface
1-12" x 12" - 5'0"
Replace Wooden with Steel Wedges, shown below
SECTION C-C
Blocking
1 Pl. 2½" x ⅜" - 5'2"
3-24" Is, 120 lb. 40'0"
6" x 8" - 5'0" Ties
SECTION A-A
SECTION B-B

Fig. 91.

less than 6 ft. above mean high water, and the soil is very soft to a depth of more than 30 ft. below mean high water, where good coarse sand is found. The south shore piers of the Harlem River Bridge and a number of foundations carrying the elevated structure are in this vicinity. The bridge piers are carried down solidly to a firm bearing; the structure foundations consist of masonry piers to mean low water, supported on piles. The softness of the soil causes it to have a tendency to move toward the river, as shown by the existing foundations, the tops of which, in the course of time, have shifted an appreciable distance, making it necessary to disconnect the columns from the foundations and reset them. Although the existing foundations might have been sufficient to carry the additional loading of the new structure, it was not deemed advisable to add to their loading. The new structure, therefore, was supported on new and independent foundations. It was desired, however, to construct these new foundations in such a manner that they would be able to resist the tendency to move toward the river, and so that, at the same time, their construction would not disturb the existing structure or bridge foundations.

The details of the designs and the location of these three foundations are shown on Plate XI. All three foundations consisted of concrete carried down to solid ground and encased in a steel sheet-piling. Two of them were circular. The third was intended to replace an existing foundation without removing the latter. It was built, therefore, around the existing foundation, but on account of various sub-surface obstructions, it had to be divided into two parts. The obstructions consisted of a 48-in. gas main curving around and into a steel-lined shaft, 10 ft. in diameter, from which it was carried in a tunnel across the river, an old cribwork bulkhead, a pilework bulkhead, and six submarine cables, supplying electric current for the Borough of The Bronx.

The construction of these foundations was done in the following manner: The steel piling for a foundation was first driven to the full depth. The excavation was then made, without attempting to remove the water, in fact, it was desired to retain the water in order to lessen the pressure on the piling and decrease the possibility of material flowing from the outside into the excavation. Finally, the concrete was placed, through the water.

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
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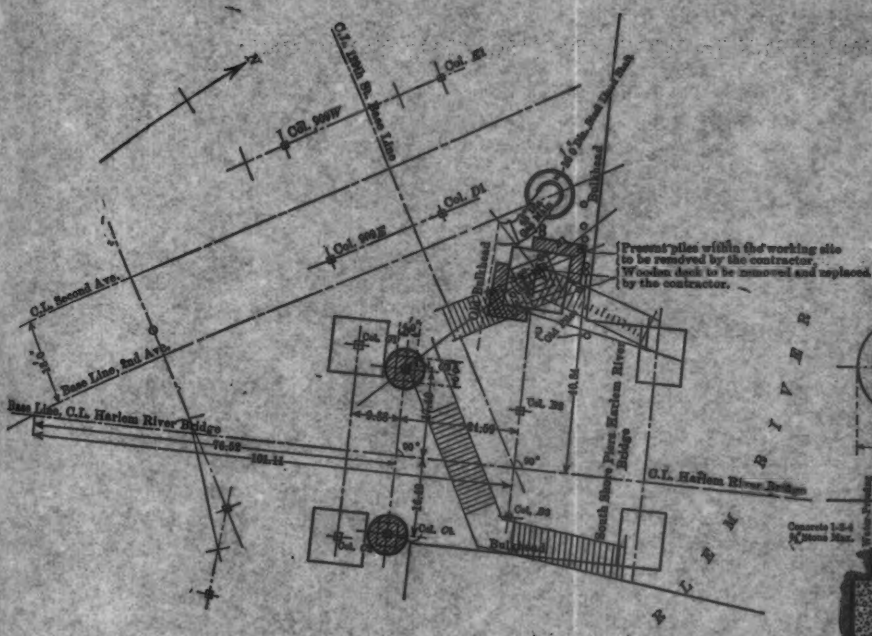
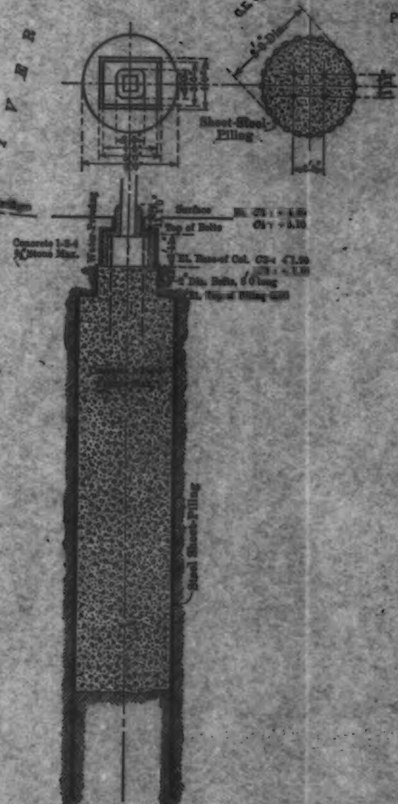
GARDINER AND JOHANNESSON ON
ELEVATED RAILWAY IMPROVEMENTS.

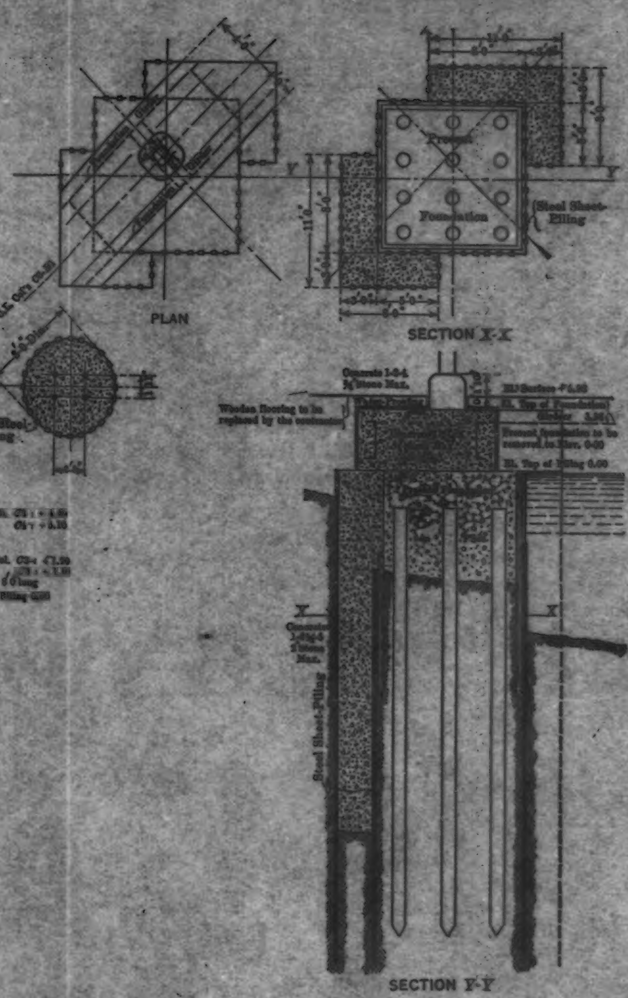


LOCATION DIAGRAM

FOUNDATIONS, SOUTH OF HARLEM RIVER BRIDGE
SECOND AVENUE LINE



DETAILS OF FOUNDATIONS FOR COLUMNS C3 AND C4



DETAILS OF FOUNDATIONS FOR NEW COLUMN "B"

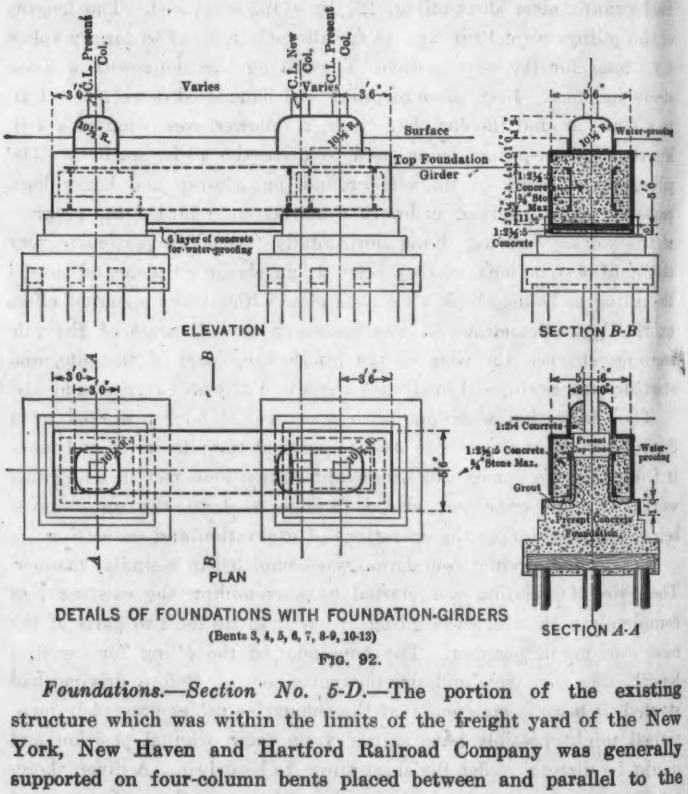
One of the cylindrical foundations was first started. On account of the foundation being partly under the existing structure, the sheet-piling could only be set in short lengths, and three lengths were used to bring it to the full depth. A pit was first excavated by hand to low-water level and shored. A wooden drum, about 8 ft. in diameter, was then set in the pit as a guide for the sheet-pile driving, and the first length of sheet-piling was set up and interlocked complete. Lackawanna steel sheet-piling, $12\frac{3}{4}$ by $\frac{1}{2}$ in., was used. The lengths of the pilings were 10 ft. and 14 ft., alternately, so as to form a splice 4 ft. long for the next section. The driving was done with a 2-ton steam hammer. Each piece of piling was hammered down about 1 ft. at a time, around the complete circle, a follower, consisting of a 4-ft. length of sheet-piling, being used to drive the 10-ft. sections. The piling drove easily in the soft ground, but rip-rap and heavy logs, probably part of former cribwork construction, retarded the progress of the work. The logs lying horizontally formed in particular very obstinate obstructions, as they acted as an elastic cushion and caused the piling to bounce back after each blow without any apparent effect on the log. Sometimes it was necessary to pull some of the pile sections, sharpen the edge of the interlocking part of the pile, and start driving again, and finally the log would give way rather suddenly.

The excavation was done with a clam-shell bucket worked by a derrick, which had also been used to suspend the pile-driver and move it from place to place. No serious difficulties were met in doing this work. The concrete was placed through a chute, the water never being removed during the operations of excavation and concreting.

The second circular foundation was completed in a similar manner. The third foundation was started by surrounding the existing pier completely with steel sheet-piling, in order to tie the two parts of the new foundation together. The remainder of the piling for the first length was then set, and driving commenced. Before driving had started, it became apparent that the submarine cables previously mentioned might possibly have shifted from their original position and might be directly under the sheet-piling to be driven. A diver, therefore, was employed to locate the cables and remove them, if necessary. As some of the cables had been placed prior to the shaft sinking for the gas main, they were buried from 8 to 10 ft. below earth and rip-rap. The rip-rap was removed by the diver, who placed it in a bucket which

was hoisted from the top, the soft soil was removed by a water jet, and the cables were replaced so as not to interfere with the driving of the sheeting.

The piling was driven until hard bottom was encountered, which was 34 ft. below mean high water on the land side and 44 ft. on the river side. This difference was probably due to the disturbance of the soil on the river side during the shaft sinking for the gas main.



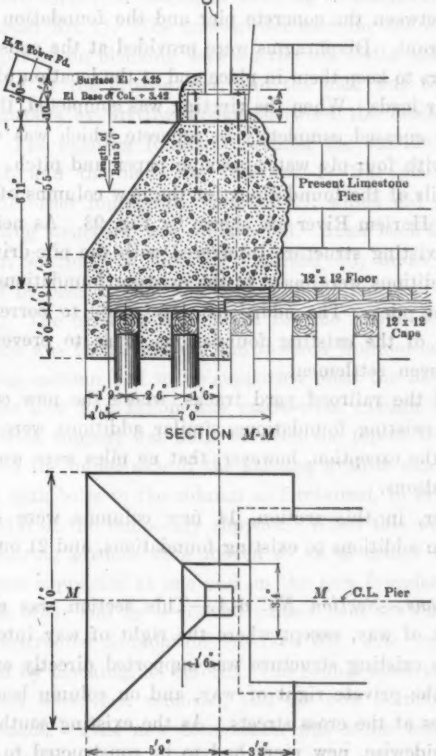
DETAILS OF FOUNDATIONS WITH FOUNDATION-GIRDERS

(Bents 3, 4, 5, 6, 7, 8-9, 10-13)

FIG. 92.

Foundations.—Section No. 5-D.—The portion of the existing structure which was within the limits of the freight yard of the New York, New Haven and Hartford Railroad Company was generally supported on four-column bents placed between and parallel to the yard tracks. Each column was carried on a separate concrete foundation with a granite cap, and supported on piles. The new columns were on a line with the existing bents and between a pair of existing columns, but generally so close to one of them that a new foundation

could not be placed without interfering with the existing one. In addition to this, the driving of piles for new foundations was out of the question on account of the interference with the traffic in the railroad yard and the lack of head-room under the existing structure.



DETAILS OF NEW FOUNDATIONS
NEAR HARLEM RIVER
FIG. 93.

The method used for supporting the new columns, therefore, was to bridge the space between the existing foundations with two foundation girders supported by the existing foundations, and to connect the column to these girders. Fig. 92 shows the details of these founda-

tions. Each of the foundation girders, which were 3 ft. deep, consisted of a web-plate and a single top and bottom flange angle. They were spaced generally 3 ft. apart, which brought them outside of the cap-stone of the existing foundations. The concrete of the existing foundations was trimmed down, so as to make room and provide a seat for the foundation girders. After the girders had been set in place, the spaces between the concrete pier and the foundation girders were filled with grout. Diaphragms were provided at the ends of the foundation girders to keep them in place and at the location of the columns to carry their loads. When the riveting was completed, the foundation girders were encased completely in concrete which was water-proofed all around with four-ply water-proofing paper and pitch.

The details of the foundations for the new columns of the first two bents of the Harlem River are shown by Fig. 93. As neither railroad tracks nor existing structure interfered with the pile-driving at these locations, additions were made to the existing foundations, which were supported on piles. The additions were made to correspond to the construction of the existing foundations so as to prevent, as far as possible, uneven settlement.

North of the railroad yard tracks, where the new columns came close to the existing foundations, similar additions were made to the latter, with the exception, however, that no piles were used to support these foundations.

Altogether, in this section, 14 new columns were supported on girders, 12 on additions to existing foundations, and 21 on independent foundations.

Foundations.—Section No. 6-A.—This section was entirely on a private right of way, except where the right of way intersected cross streets. The existing structure was supported directly on brick piers throughout the private right of way, and on column bents set inside the curb lines at the cross streets. As the existing south-bound track was moved sidewise, new piers had to be constructed to support this track, in addition to the new foundations required to support the additional structure. Altogether, 53 new piers and 205 new foundations were constructed in this section.

The existing bents at the cross streets were supported on four columns set on the sidewalk parallel to the curb. The new supporting columns of the bents were on line with the existing columns, but gen-

erally at different locations. Plate XII shows the method of shoring used in rebuilding the foundations. First, four shoring girders, consisting of 24-in. Bethlehem, I-beams, or two plate girders, 4 ft. deep, were placed parallel to the bent, two on each side of the columns, and approximately centered on one of the existing tracks. These shoring girders were supported on timber blocking at the points marked *A*, *B* and *D* (Sketch No. 1). The cross-girder carrying the existing track was then supported on blocking between the top of the shoring girders and the bottom of the cross-girder, and the load was transferred to the shoring girders by driving wedges between the blocking and the cross-girder. The existing foundation under the outside column was then removed, and the new one (marked 1) was placed. At the same time, the other foundation (marked 2), which did not interfere with the existing structure, was constructed. After back-filling around Foundation 1, so that the blocking (marked *C*), under the shoring girders, could be placed; this was done and the blocking (marked *B*) was removed. The existing foundation for the inside column of the supported track was then removed, and the new foundation was placed. As the existing column had to be supported until the new steel could be erected, the new foundation was made wide enough to carry this column. A brick support was built between the top of the permanent foundation and the under side of the existing granite cap-stone, which was attached with bolts to the column and retained, in order to facilitate the removal of the temporary support. When this work was completed, the shoring girders were moved so as to center on the other track, and were supported at one end on the new foundation (marked 2) and at the other end by a framed timber bent set on the base of the newly constructed foundation (marked 3). The track structure was supported by blocking, as before. The foundations under the two shored columns were then removed, and the new permanent foundations were placed. This new foundation was made wide enough to support the existing columns temporarily.

At 138th Street, where a new subway was being built, it was desired to carry the foundation down to solid rock. The rock surface at this point was very irregular, so that, though two of the foundations were carried only 17 ft. below the surface of the street, three of them had to be carried down about 39 ft., which brought them below the subgrade of the subway.

Fig. 94 is typical of the foundation work within the right of way. The light lines represent the piers supporting the existing structure, and the heavy lines represent the new work. The new pier was built to support the existing south-bound track, which was moved sidewise, and the foundations supported the new structure. The new piers were generally carried down to the bottom of the existing piers, about 6 ft. below the surface, but, in addition to this, before the piers were built, an excavation was made along the property line under each pier, 3 ft. wide, 16 ft. long, and about 12 ft. deep below the curb level of the nearest street, and filled with concrete, in order to prevent possible undermining of the piers, if the adjoining houses should be rebuilt.

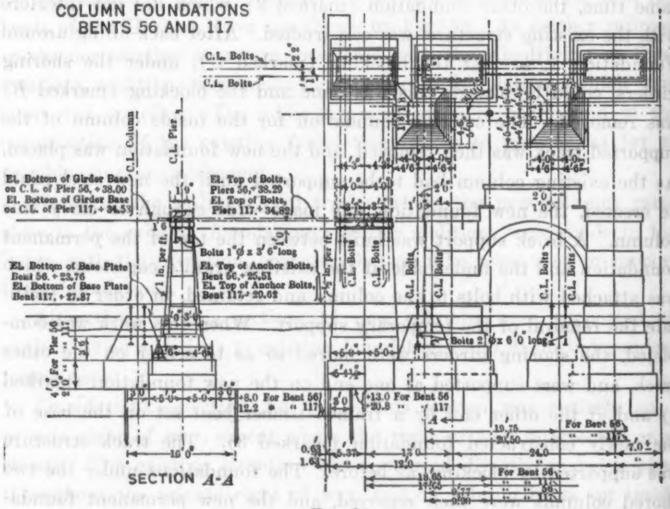


Fig. 94.

The foundation and the piers on the west side of the right of way were completed first, starting from the south end and going north. The remaining foundations on the east side were then constructed, starting at the center of a block and working in both directions in order to facilitate the handling of the materials in the limited working space.

The forms for the piers were detachable, and each side was in three sections and secured in place by timber yokes. On account

PLATE XII.
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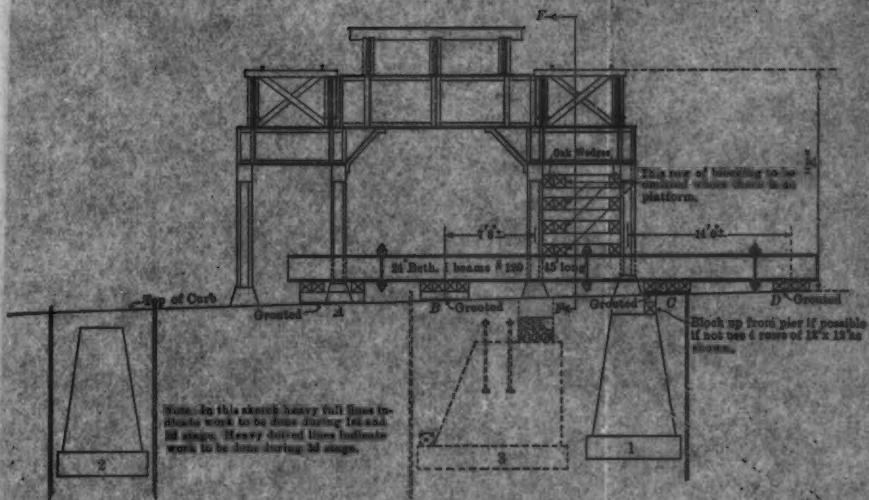
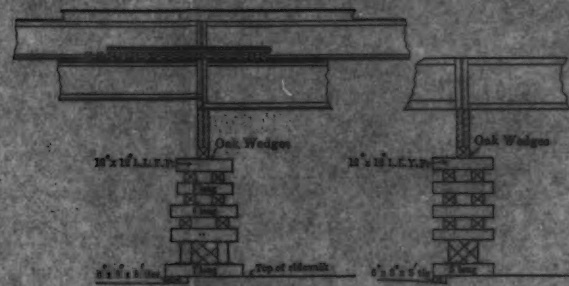
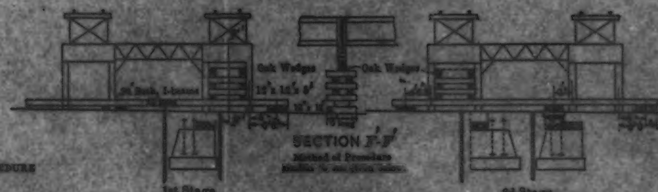


SKETCH NO. 1.—1st 2d AND 3d STAGE, LOOKING NORTH.



SECTION E-E
Where there is a platform.
Bents 39-40-60-61-125 & 126.

SECTION F-F
Where there is no platform.



SECTION F'-F'
Method of Procedure

SKETCH NO. 3.—TYPICAL SHORING FOR BENTS 129-130 AND 136, LOOKING NORTH.

METHOD OF PROCEDURE

1st Stage

Shore up North-Bound Cross-Girder as shown in sketch No. 1, placing the blocking under the 24 I-Beams at A, B, and D. Excavate for and Build foundation No. 1.

2d Stage

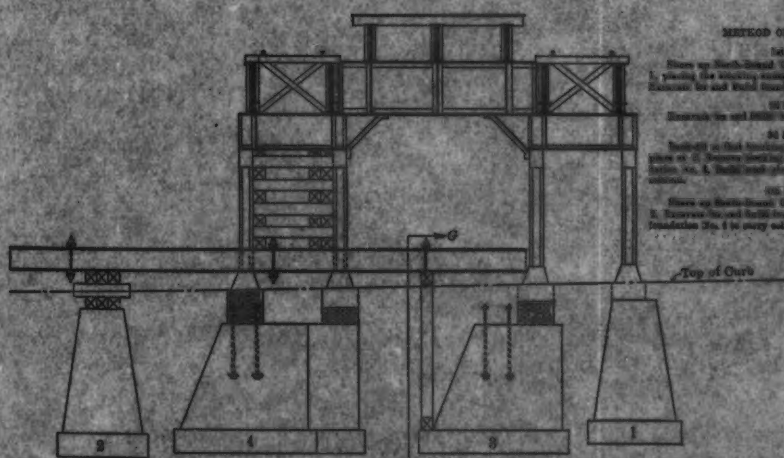
Excavate for and Build foundation No. 2.

3d Stage

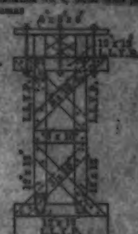
[illegible] blocking under 24 I-Beams [illegible] in place at C. Remove blocking at B. Excavate for and Build foundation No. 3. Build brick pier on foundation No. 3 to carry column.

4th Stage

Shore up South-Bound Cross-Girder as shown in sketch No. 2. Excavate for and Build foundation No. 4. Build brick pier on foundation No. 4 to carry column.



SKETCH NO. 2.—FINAL STAGE, LOOKING NORTH.



SECTION G-G

SHORING FOR FOUNDATIONS
IN STREETS
BETWEEN 134TH AND 145TH STREETS

of the large quantity of concrete to be mixed, a concrete mixer was used for the construction of the piers. The mixer was mounted on a platform which also carried the implements for hoisting and dumping the concrete. The platform was moved on rollers, using a drum on the hoist for motive agent.

Steel Erection.

The greater part of the work under the Manhattan Elevated Railway Improvements was in public streets, often congested with traffic. The steelwork, the total weight of which was about 50 000 tons, therefore, could not be stored in large quantities, or for any length of time, at the points where it was required for erection. The greater part of the steel came over the Pennsylvania Railroad and the Central Railroad of New Jersey, and was unloaded and stored in the railroad terminal yards in New Jersey, at Greenville and Communipaw, respectively. The receiving points for the steel in New York were the Company's Yard at 128th Street and the Harlem River, the dock front of which was increased by leasing adjacent property, and a dock front at Perry Street and the North River was leased for the purpose. All the steelwork for the up-town sections was delivered at the 128th Street dock, and that required for the down-town sections at the Perry Street dock. As the material was needed, it was lightered from the railroad yards to the Company's docks, where it was stored temporarily until it could be trucked away. The trucks were loaded at each dock with a derrick erected for the purpose. The material was trucked to the site of the work and deposited in the street until it could be erected, which generally meant that it remained in the street only for a day or two. In this manner, the congestion of the streets due to storage of material was reduced to a minimum.

Steel Erection.—Section No. 1.—Fig. 95 is a bird's-eye view of the greater portion of Section No. 1 after completion. The view was taken from Chatham Square, and shows the City Hall Branch in Park Row to the right and the South Ferry Branch in the New Bowery to the left. Fig. 96 shows the general layout of the old tracks at Chatham Square. The first work done was the erection of the steel for the west platform of the new City Hall Branch Station at the west side of Chatham Square. The steelwork for this platform and the adjacent tracks is shown on Fig. 96. When the station was

completed, and the new west track was connected to the existing south-bound track, the south-bound service was diverted from the existing station just north of Chatham Square to this new station and track. The old south-bound track was then removed, so that connection could be made to the new north-bound track east of the new platform. When this work was completed, the old station was abandoned and the structure carrying the disused portion of the tracks was dismantled, making room for the construction of the new two-track structure, connecting to the new upper deck of the City Hall Station.

The erection of the new structure in the New Bowery was started at the south end, where the ramp commenced at the grade of the present structure. The erection of this structure involved only minor changes in the existing tracks. The new structure in Park Row was erected with a traveler, starting at Chatham Square and going through on the upper deck to City Hall Station. When the new structure had risen sufficiently above the lower deck, it was carried immediately above the existing tracks. Before the traveler erected the upper-deck girders, the new columns were erected, and the lower-deck cross-girders were either reinforced or replaced. Fig. 97 shows the shoring of the structure used to carry out this work. Two four-post bents were placed at each side of the structure and supported two shoring girders which carried the track stringers and the track while the old columns and cross-girders were being removed and the new ones placed. The erection of the new work was done with gin-poles, and, in order to facilitate their erection, the new lower-deck cross-girders were shipped bolted and, in the beginning, raised member by member and riveted in place. As the work progressed, it was found that they could be raised complete without serious interference with the traffic in the street, and, therefore, they were riveted together completely before erection with the exception of the top flange, which had to be erected separately, as it extended above the top of the track stringers, the ends of which were so close together that the flange of the cross-girder could not pass through the intervening space.

Steel Erection.—Section No. 2-A.—Whenever the erection of the new structure necessitated interference with the train service, the method of procedure was carefully worked out in advance, so as to insure at all times during the progress of the work safe and continuous

FIG. 95.—CHATHAM SQUARE, LOOKING SOUTH.

FIG. 96.—CHATHAM SQUARE AT THE BEGINNING OF RECONSTRUCTION.

FIG. 97.—SHORING IN PARK ROW.

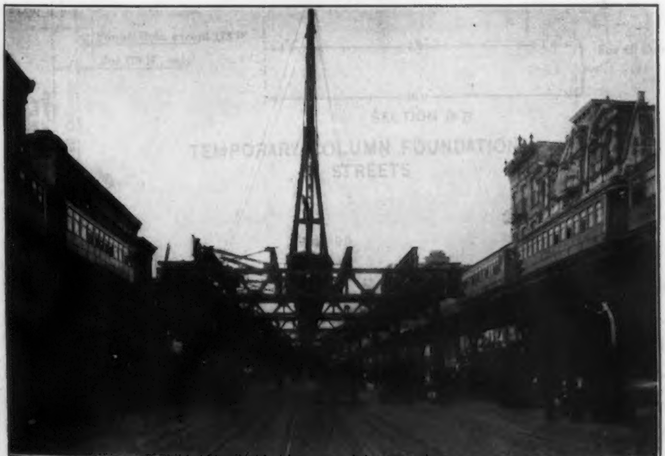
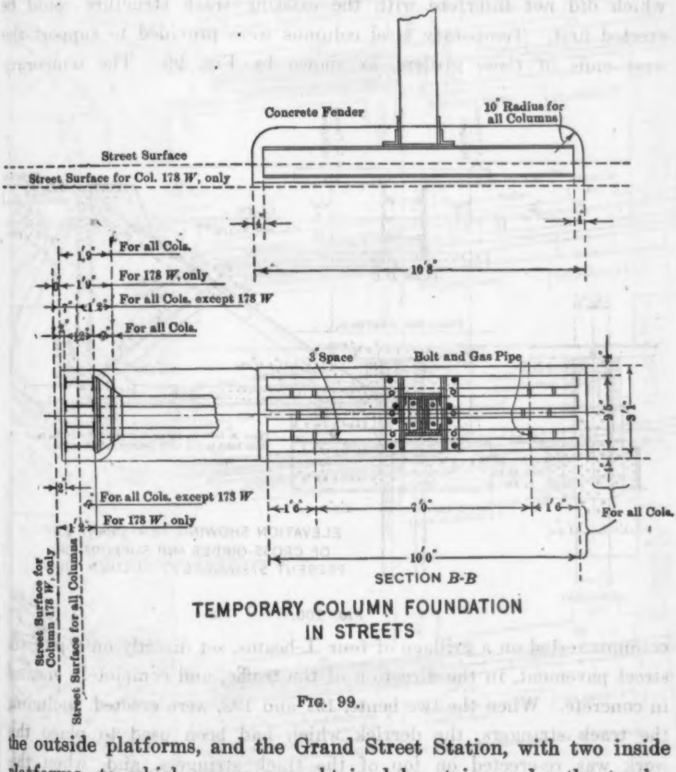


FIG. 98.—NEW AND OLD STRUCTURES IN THE BOWERY.

operation of trains and station facilities. Plate XIII shows the method of procedure worked out in advance of, strictly followed in the erection of, the new steel structure in Section No. 2-A. The plan marked "Original Condition" shows the platforms and tracks which existed prior to the reconstruction work. It shows the old Chatham Square Station, with the pocket tracks, the Canal Street Station, with



TEMPORARY COLUMN FOUNDATION IN STREETS

FIG. 99.

the outside platforms, and the Grand Street Station, with two inside platforms, to which access was obtained by stairs under the tracks. The new steelwork through the old Grand Street Station could not be placed until the new station had been erected, as the existing platforms and station building, which covered the entire space between the tracks, had to be maintained. The steel erection, therefore, was commenced north and south of the old Grand Street Station.

South of Grand Street Station, the two bents, numbered 191 and 192, were first erected with a derrick set up in the street. As the new permanent columns on the west side of the street were to be on the sidewalk under the existing structure, and, therefore, with the cross-girders supported by them, would interfere with the existing track structure, the new cross-girders were designed so that those portions which did not interfere with the existing track structure could be erected first. Temporary steel columns were provided to support the west ends of these girders, as shown by Fig. 99. The temporary

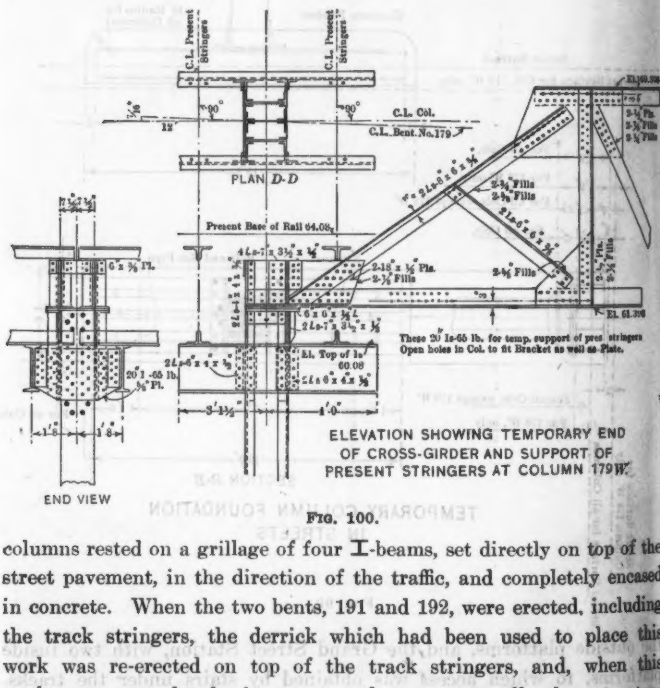


Fig. 100.

columns rested on a grillage of four I-beams, set directly on top of the street pavement, in the direction of the traffic, and completely encased in concrete. When the two bents, 191 and 192, were erected, including the track stringers, the derrick which had been used to place this work was re-erected on top of the track stringers, and, when this work was completed, it was used to erect all the structure south of this point. At and south of Canal Street there were several bents which were to be supported permanently on sidewalk columns. As the new structure was several feet above the present one, parts only of the cross-girders were erected, and temporary columns in the

PLATE XIII.
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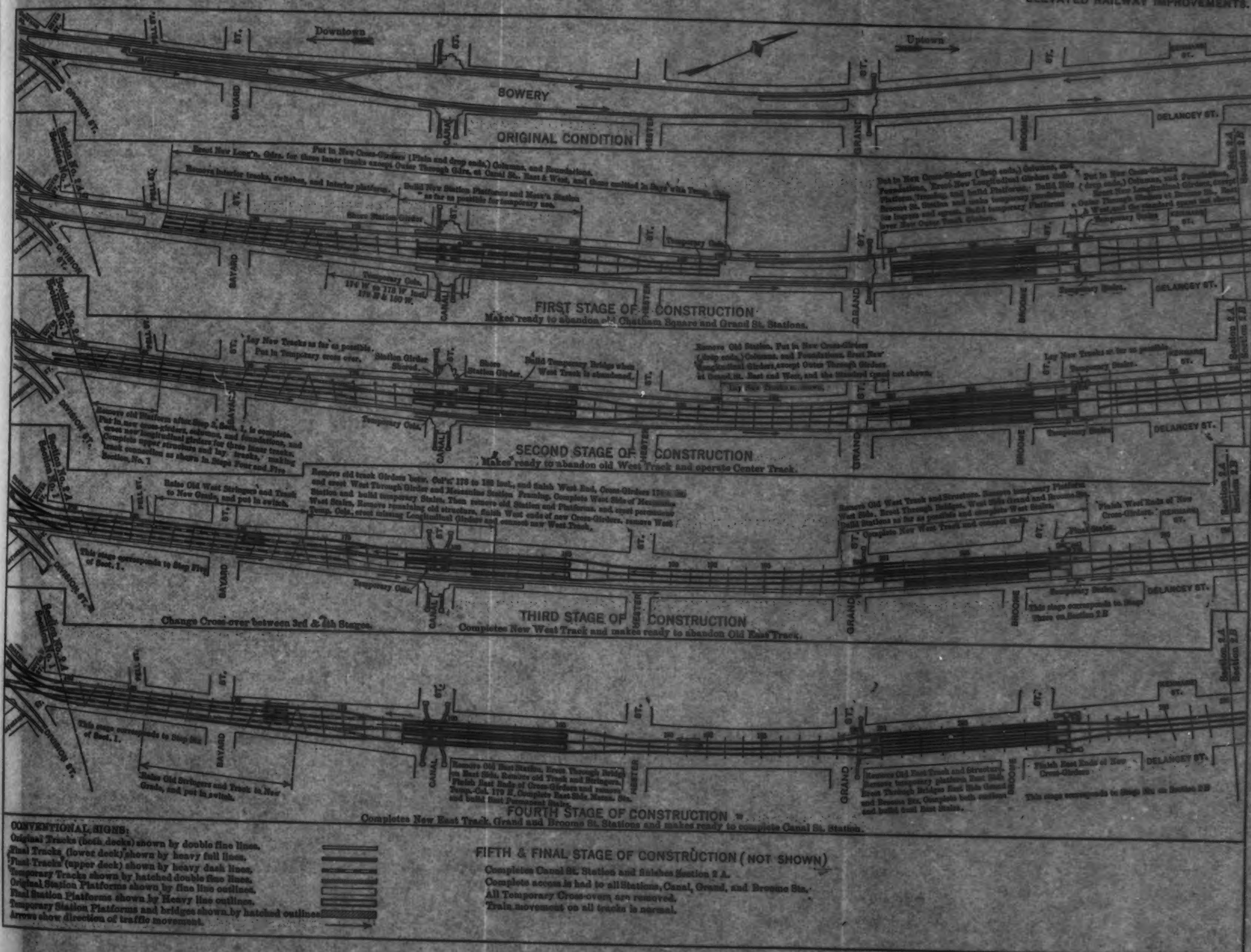


roadway were provided to support these cross-girders. At one point —the south side of Canal Street—the street-railway tracks which curved around from the Bowery to Canal Street, did not permit the placing of a temporary column in the street. The cross-girder, there-

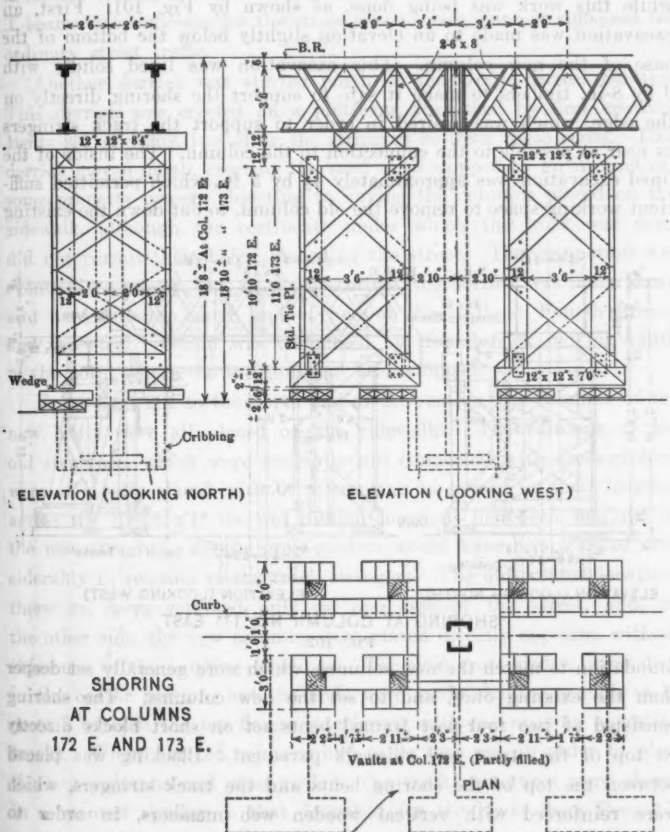
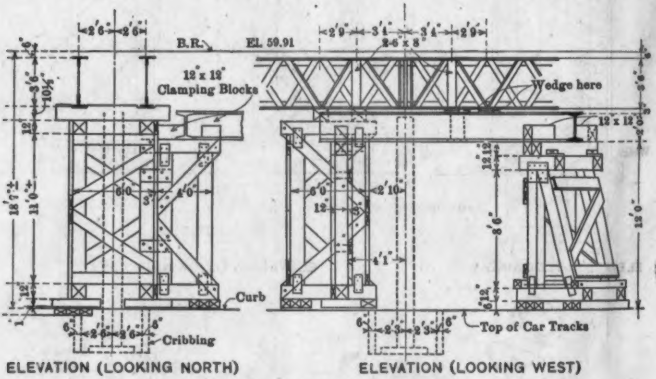


FIG. 101.

fore, was provided with a temporary shallow end which was supported on the permanent column on the sidewalk, as shown by Fig. 100. To support the existing track stringers, a temporary bracket was riveted to the new column shaft, as shown. From Bent 173 south, the new

columns were all on the sidewalk, and replaced the existing columns. The new lower-track level approached the level of the existing tracks, and the cross-girders, therefore, were designed to be placed below and without interfering with those tracks. The track stringers were shored while this work was being done, as shown by Fig. 101. First, an excavation was made to an elevation slightly below the bottom of the base of the new column. This excavation was lined solidly with 6 by 8-in. timbers to make it safe to support the shoring directly on the edge, which was desired in order to support the track stringers as near as possible to the connection to the column. The inside of the lined excavation was approximately 4½ by 5 ft., which permitted sufficient working space to remove the old column, to cut down the existing



SHORING AT COLUMN NO. 171 EAST
FIG. 102.

foundation to match the new columns, which were generally set deeper than the existing ones, and to set the new columns. The shoring consisted of two four-post framed bents set on short blocks directly on top of the street and sidewalk pavement. Blocking was placed between the top of the shoring bents and the track stringers, which were reinforced with vertical wooden web members, in order to transmit the web stresses directly to the shoring. The load was transferred from the existing column to the shoring by driving wedges between the bottom of the shoring bents and the blocking in the street. Special shoring had to be provided at certain places where street-car tracks interfered with the usual method. Fig. 102 shows the method of shoring at Bent 171, where a street-railway

track curves into the Bowery. On one side of the column a standard shoring bent was used; on the other side two shoring bents were placed so as to clear the car track, and the structure was carried by two 24-in. I-beams, resting on the shoring bents. The clearance below these I-beams was sufficient for the street cars to pass, but not sufficient for ordinary street traffic.

Another derrick was started north of the Grand Street Station. This derrick was erected on a platform, furnished by first erecting Bents 201 and 202, including the stringers between these bents. This derrick moved north, erecting the steel until stopped by an injunction, some property owners objecting to having the columns placed on the sidewalk, although the certificate under which the work was done did not permit their being placed in the street. The injunction was eventually removed, but, in the meantime, the derrick was taken down and moved to the north end of Section No. 2-B, at Fourth Street, and the steel erection was completed by this derrick moving south to the point where the erection had been stopped.

From Bent 201 to the north end of the section, the columns of the new bents were all placed on the sidewalks. The columns of the old structure, which were generally not connected with cross-girders, were originally placed without attempting to square up their location across the street. If the old column locations had been retained in the new structure, all the cross-girders would have been skewed considerably in relation to the track stringers. The old column locations therefore, were retained only on one side of the street, and, on the other side, the new columns were placed directly opposite, without reference to the existing locations. The new cross-girders were deep in the center portion, but the ends were made shallow enough to be erected without the top flange interfering with the existing operating tracks. Where the old columns had to be removed, in order to place the new columns and cross-girders, the track structures were shored in a manner similar to that described for the bents from Bent 173 south. On the other side of the street, the new cross-girders intersected the existing track stringers, which, therefore, had to be cut prior to the erection of the cross-girders. It will be remembered that the new foundations for the columns supporting these new cross-girders, had been constructed in advance of this work. These foundations were uncovered sufficiently to set the new columns, and the pit

was lined with 6 by 8-in. timbers, as described. The new column was then set in place, and a four-post framed shoring bent was built around it, as shown by Fig. 103, supported on blocking laid on the street surface. The shoring bent was made to carry the load of the stringers by driving wedges between the top of the blocking and the bottom of the shoring bent. The work to be done consisted of cutting the near-side track stringers to make room for the new cross-girders. The far-side track stringer was not affected, as the cross-girder did not extend far enough to interfere with it. After the shoring bent was

REPLACEMENT OF COLUMN

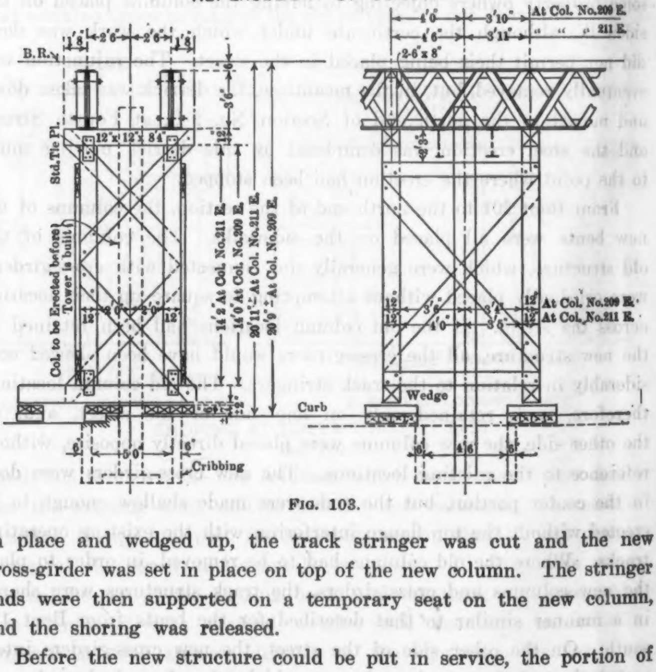


FIG. 103.

in place and wedged up, the track stringer was cut and the new cross-girder was set in place on top of the new column. The stringer ends were then supported on a temporary seat on the new column, and the shoring was released.

Before the new structure could be put in service, the portion of the structure occupied by the existing Grand Street Station had to be placed. This could not be done until the new Grand Street Station had been sufficiently conpleted and provided with temporary platforms to serve the existing tracks. This was accomplished by the traveler placing the new outside track stringers (which could not be

put in their permanent position, as they would then interfere with the running tracks) in a temporary position adjacent to the running tracks. As the new track level was purposely placed at the correct elevation above the existing track level, the new track stringers in their temporary positions were just suitable for supporting the temporary platforms. The new mezzanine station building of the Grand Street Station at the Broome Street end was sufficiently completed to be put in service, and was connected to the temporary platforms. The old Grand Street Station was then abandoned and demolished, and the new steel structure was completed throughout the section sufficiently to lay the center track. The new west island platforms of the station at Grand Street and Canal Street were then completed. At the Canal Street Station it was originally intended to gain access to the new platforms by a temporary bridge from the old platform after the south-bound track had been abandoned. It was found, however, that it would be possible to place a temporary stairway at the east side of the roadway close to the curb at the north side of Canal Street. This stairway, therefore, was placed connecting to the new mezzanine station at Canal Street, which was sufficiently completed to be placed in service. The scheme was to use temporarily the new center track for south-bound service, so that the existing south-bound track could be abandoned and the new structure on the west side completed. The new permanent connection, therefore, was made between the new center track and the local south-bound track to City Hall at Chatham Square just south of Section No. 2-A, and a temporary cross-over was provided between the center track and the new south-bound local track to South Ferry, just south of the Canal Street Station. This local track to South Ferry rises above the grade of the existing tracks and continues on the upper-grade structure, which had been constructed as part of Section No. 1 through the New Bowery and had been completed ready for operation at this time. At the same time the new center track had been completed through Section No. 2-B to Fourth Street, and the local traffic was temporarily carried on the newly constructed tracks from Fourth Street to Chatham Square. This left the existing structures on the west side of the Bowery free to be removed, which was done, and the new structure was completed on this side of the street. This work included the erection of the sidewalk columns and the com-

pletion of the cross-girders, the erection of which had been interfered with by the existing structure, and also the removal of the temporary columns on the west side of the roadway. When this work had been completed and the new south-bound track had been laid, the temporary cross-over south of Canal Street was removed, and the permanent connection was made to the new south-bound track to South Ferry. The south-bound traffic was then diverted from the center track to the new permanent south-bound track and the north-bound traffic between Chatham Square and Fourth Street to the center track, which involved the completion of the permanent track connections at Chatham Square and the placing of a temporary cross-over south of Canal Street to connect the new upper-deck track from South Ferry with the center track. The work remaining to be done to complete the structure on the east side of the Bowery was then completed, the temporary cross-over south of Canal Street was removed, and the local north-bound service was transferred to its permanent track.

Steel Erection.—Section No. 2-B.—In Section No. 2-B, which extended, on the Bowery, from the north end of Section No. 2-A, near Delancey Street, to Fifth Street, the new structure was placed in the middle of the roadway; the existing structure was on the sidewalk. There was, therefore, practically no interference with the existing tracks during the steel erection, except near Fifth Street, where a connection was made to the existing structure, which, at this point, is in the roadway. Following a similar procedure to that described under Section No. 2-A, two bents and their track stringers were erected near Fourth Street with a derrick set up in the street. The derrick was then erected on this platform and moved south, erecting the steel structure as it proceeded.

Later, another derrick was set up on the same platform to erect the steel north of this point. The new track level was several inches higher than the existing level, and, as it became necessary to make temporary connections with the running tracks, these were raised to make the connections on a proper grade. The raising was done by placing four-post shoring bents under the track stringers, and jacking and wedging the track stringers to the proper level after the column had been disconnected.

Fig. 98 shows the erection in progress in the vicinity of the Houston Street Station.

Steel Erection.—Section No. 3.—The first work of steel erection on this section was the reinforcement of the column tops, as described under the heading, "Steel Design." The reinforcement of existing lattice cross-girders between 33d and 35th Streets was commenced soon thereafter. It consisted of replacing the existing lattice members with web-plates. While this work was being done, the track stringers were carried on shorings, as shown by Fig. 104. On each side of the columns supporting a cross-girder was placed a four-post shoring bent with the posts tapering. The shoring bents rested on blocking laid on the surface of the street; this blocking was leveled up with wedges, and a lean, liquid mixture of sand and cement was poured to fill the space between it and the street surface. On top of each pair of shoring bents was placed two 24-in. I-beams crosswise under the structure. The track stringers were supported by blocking on these I-beams, and the load was transferred to the shoring by wedging under the track stringers. The reinforcing web-plates, which were generally about 14 ft. long, were raised to their proper level between the building line and the structure, and were then gradually worked in between the ends of the track stringers. At the 34th Street Station, where the station structure interfered with the placing of the plates in this manner, they were erected in shorter pieces between the tracks.

After this work was completed, the cross-girders between 12th and 16th Streets were reinforced. At this location the existing cross-girders were made with two end pieces and a center piece. The existing end pieces were removed and replaced with new ones, and the center piece was reinforced with a new web-plate. In addition to this the center-track stringers were remodeled to change the existing connections to the cross-girders to seat the connections. While this work was going on, a traveler was erected just north of 35th Street to place the new track stringers and cross-girders north of this point. In order to place the new cross-girders the existing track stringers were disconnected and carried on shoring until the erection of the new steelwork was completed. Two bents were shored at the same time, and the shoring was moved forward as soon as the work permitted its release. The work of erecting the steelwork for the hump stations at 9th, 23d, 42d, and 106th Streets was done by travelers starting at one end of the

approach and moving forward, erecting all steelwork, except some for the express platform, which generally was erected with a jinnywink operating behind the traveler. Fig. 125 shows the erection of the steelwork for the express station and track at the 9th Street Station as well as the derrick and the jinnywink in operation.

Steel Erection.—Section No. 4-A.—The steel erection for this section, which comprised the Second Avenue Line from Chatham Square to 116th Street, involved mainly the placing of new track stringers for the center track, which work was done with traveling derricks moving forward on the stringers set by the traveler. At the 86th Street Station, it was originally intended to have no express station, but after the center track stringers had been designed and fabricated to be placed on the level of the existing tracks, the Public Service Commission ordered an express station to be built at this point. This station is of the hump type. Another hump station was constructed at 14th Street, and a mezzanine station was built at 42d Street. The existing structure at this point was not sufficiently high to provide head-room under it for a mezzanine station. The track structure, therefore, was first raised to the necessary height, which amounted to a maximum of 15 in. The raising of the structure was accomplished by placing blocks between the top of the cross-girders and the under side of the lips of the track stringers. The reconstruction of the station was commenced on the west side of the structure, and necessitated the abandonment of the existing south-bound track throughout the portion to be constructed. The south-bound traffic was turned into the existing center track north and south of the station, and, in order to maintain access to the station, a temporary platform was constructed over the abandoned track adjacent to the existing platform and extending north for 260 ft. from the center of 42d Street. In order not to interfere with traffic, the timber bents supporting this platform were cut and framed in advance, and the platform was placed one night between 1 and 5 A. M., during which time no trains were operated on the Second Avenue Line.

The south end of the old south-bound platform was then removed, the existing track girders of the south-bound track were shifted into their new positions, the south half of the new west island platform was constructed, and, in order to make this platform long enough to serve the traffic, a temporary timber extension was built at the

FIG. 104.—SHORING IN THIRD AVENUE AT 33D STREET.

FIG. 105.—STATION ON SECOND AVENUE AT 42D STREET DURING RECONSTRUCTION.

south end. Access to the platform was obtained by a stairway to the new mezzanine station, which was partly completed and connected to the street by a stairway. When this work was completed and ready for operation, the existing south-bound station was abandoned for traffic, as well as the temporary platform north of 42d Street, and the south-bound trains, still running on the center track, were stopped at the new island platform. The platform construction and the west station building were then removed, and the relocation of the south-bound track and the construction of the west island platform were completed. South-bound traffic was then run on the new permanent south-bound track, and the north-bound traffic was diverted to the center track, the new west island platform serving both tracks. This left the east side of the structure clear of traffic and permitted the relocation of the north-bound track and the completion of the construction of the east island platform. Fig. 105 shows the temporary platform north of 42d Street in place and the center track being used for south-bound service; half of the station building has been removed. The new mezzanine station is placed in the span, just south of the center line of 42d Street, and is supported by the through bridge girders shown. These girders also support the track structure and the platforms. Fig. 106 shows the southerly half of the new west island platform in course of construction.

At the 92d Street Station the existing platform was centered between the two running tracks, and was removed to make room for the new express track. To replace this platform, two side platforms were constructed, and, as the structure at this point is very high, a mezzanine station was built under the track structure. The existing cross-girders were first lengthened, and the new platforms and mezzanine were constructed. When these platforms and the stairs connecting them with the mezzanine station were completed, the old center platform was removed.

Steel Erection.—Section No. 5-A.—In this section, which comprised the Third Avenue Line between 116th Street and the Harlem River, the approach to the upper-deck express station commenced near 121st Street, and, as the express track was to connect with the upper deck of the Harlem River Bridge, it remained elevated throughout the section. Before the upper-deck structure was erected, the existing cross-girders and columns supporting the lower-deck

structure in Third Avenue between 123d and 128th Streets were replaced. In order to support the track stringers while the cross-girders and columns were being removed, two 24-in. I-beams were placed under the structure on each side of the cross-girder and supported on timber shoring bents, in a manner similar to that described under Section No. 3. As it was the intention to scrap the old material, no attempt was made to preserve the members intact, and the removal of the old structure, therefore, was facilitated by burning it, with a blaugas flame, into pieces convenient to handle. After the old columns and cross-girder of a bent had been removed, the new cross-girder (which was shipped loose, with the top flange and such web-angles as might interfere) was hoisted into a position directly below that which it was to occupy permanently, and was supported in this position temporarily by the shoring. It was then worked into place by jacking and blocking through the narrow space between the ends of the track stringers. The top flange-angles were afterward pushed in from the side of the structure between the track stringers and the ties, and the work was completed by riveting the pieces together. Whenever the stringer ends were to be remodeled, this was done in conjunction with the erection of the new cross-girders. The columns, which had detachable bases, were set after the cross-girder was in place, the procedure being first to set the loose base over the anchor-bolts and then slip the column shaft in between the base and the cross-girder. As the structure was generally jacked up a little above the final elevation, in order to carry the load of the structure on the shoring, little difficulty was experienced in putting the columns in place.

South of the 125th Street Station, the existing cross-girders and columns were retained, but the cross-girders were reinforced and the ends of the center-track stringers were remodeled. As this work was done while traffic was running over the center track, the structure had to be shored. The method of shoring is shown on Fig. 108. The shoring bents had four posts, all in one plane, and were secured to the columns by yokes. These bents were not designed originally for this special purpose, but had been used on another section. The shoring girders which, as usual, consisted of two pairs of 24-in. I-beams, rested on top of these bents and supported the track stringers.

FIG. 106.—STATION IN SECOND AVENUE AT 42D STREET. STRUCTURE ABOVE MEZZANINE.

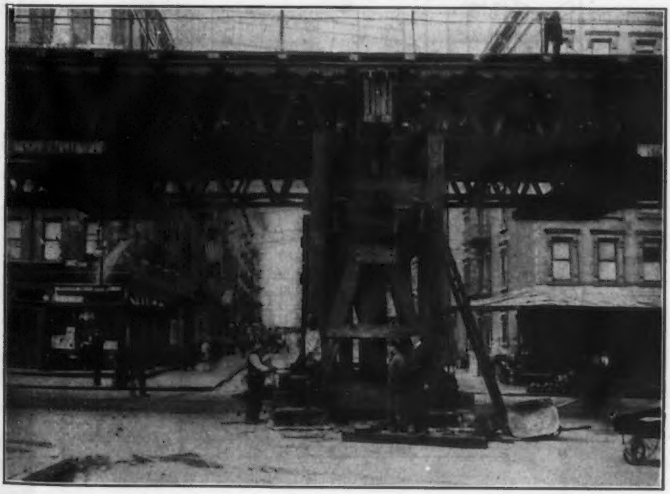
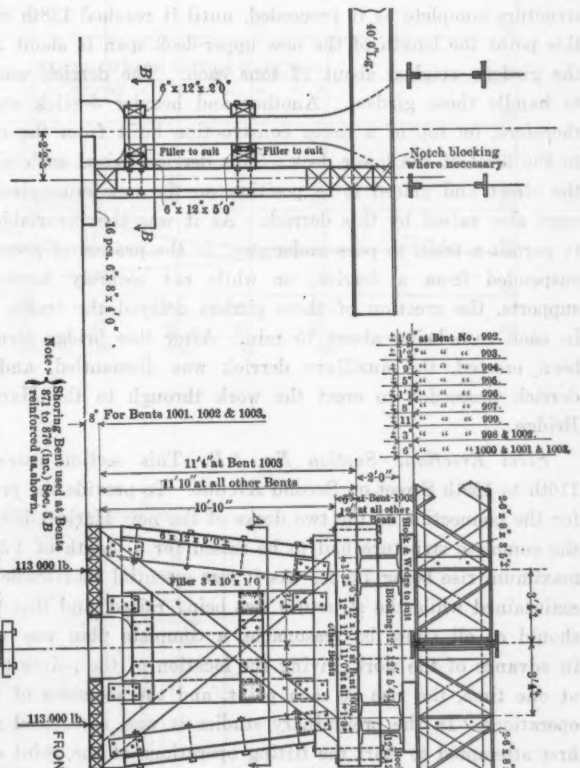


FIG. 107.—RAISING STRUCTURE IN SECOND AVENUE WITH HYDRAULIC JACKS.



SHORING OF STRUCTURE
THIRD AVENUE LINE

FIG. 108.

The derrick which erected the upper-deck structure was set up at the south end of the upper-deck approach, and erected the steel structure complete as it proceeded, until it reached 128th Street. At this point the length of the new upper-deck span is about 90 ft., and the girders weighed about 17 tons each. The derrick was not able to handle these girders. Another and heavier derrick was erected, therefore, on top of a tower construction built from the street level to the level of the lower deck. This derrick raised each girder from the street and placed it in position on the new cross-girders, which were also raised by this derrick. As it was the invariable rule not to permit a train to pass under steel in the process of erection, while suspended from a derrick, or while not securely fastened to its supports, the erection of these girders delayed the traffic, the delay in each case being about 15 min. After this bridge structure had been erected, the auxiliary derrick was dismantled, and the first derrick proceeded to erect the work through to the Harlem River Bridge.

Steel Erection.—Section No. 5-B.—This section extended from 116th to 129th Street on Second Avenue. To provide the proper grade for the connection to the two decks of the new Harlem River Bridge, the complete structure had to be raised for a length of 1 387 ft., the maximum rise being 7.5 ft. As it was essential that traffic should be maintained while the structure was being raised, and that the grades should at all times be reasonable, a complete plan was worked out in advance of the work, giving the location of the points to be raised at one time, the rise at each point, and the sequence of the lifting operations. In the preliminary studies it was, as seemed reasonable, first attempted to start the lifting operations at the point of the line where the final rise was greatest, that is, at the middle of the portion of the structure to be raised, but it was soon found that no satisfactory grade could be maintained starting at this point. The scheme decided on is shown on the upper part of Plate XIV. The lifting operations were started at the south end of the new grade, and usually five bents were raised during one operation. The maximum rise during any one operation was limited to 6 in. The second lift was partly superimposed on the first one, but extended over three additional bents, and the work was continued in this manner until five operations were completed, and the structure for about half the length had been raised

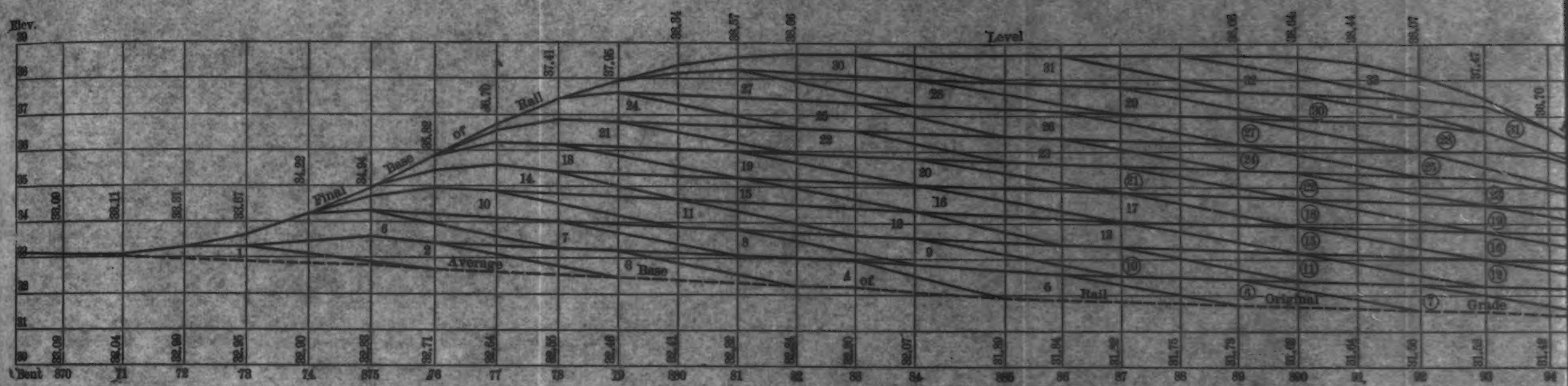
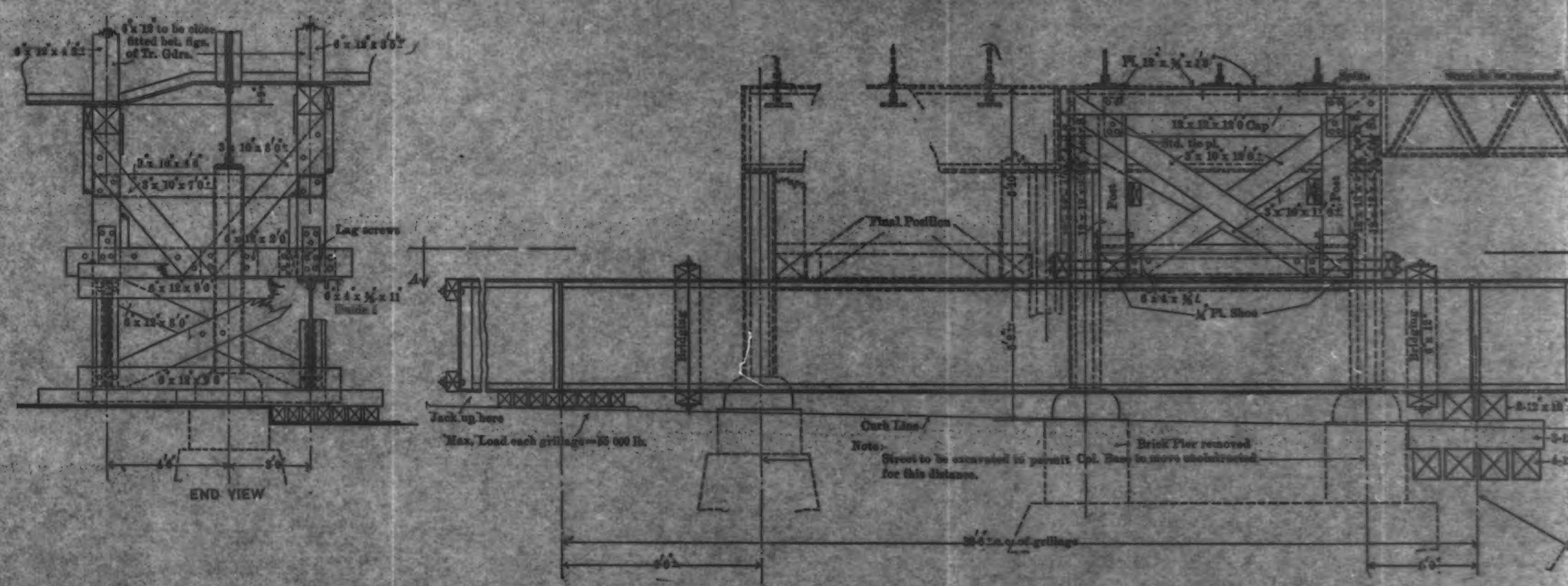


DIAGRAM OF SUCCESSIVE DAILY JACKING OPERATIONS



SHORING OF STRUCTURE, THIRD AVENUE LINE

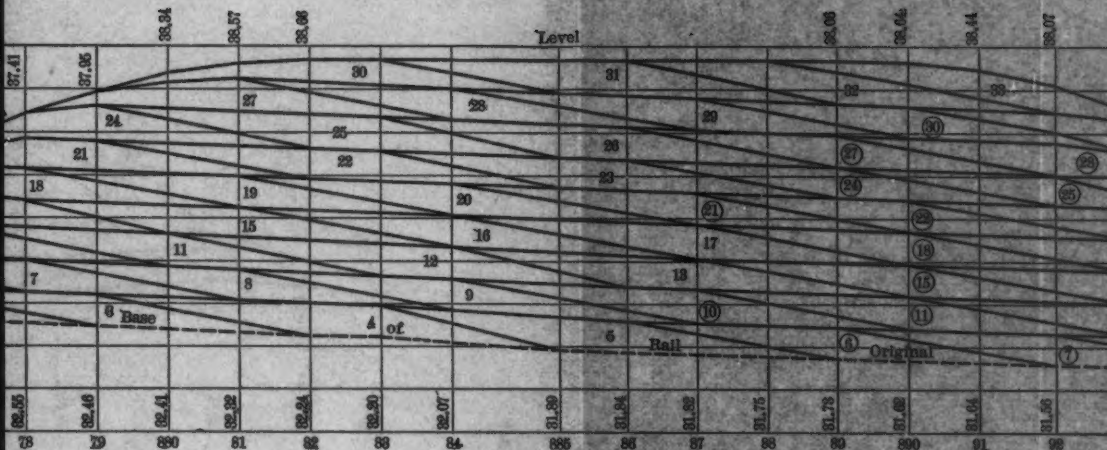
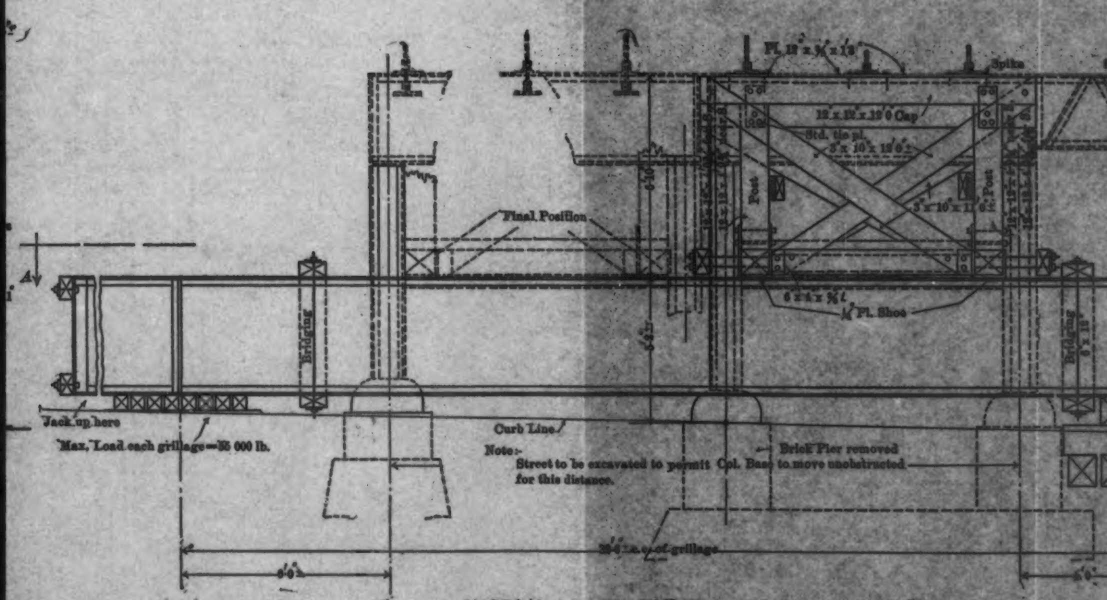


DIAGRAM OF SUCCESSIVE DAILY JACKING OPERATIONS



SHORING OF STRUCTURE, THIRD AVENUE LINE

PLATE XIV.
TRANS. AM. SOC. CIV. ENGRS.
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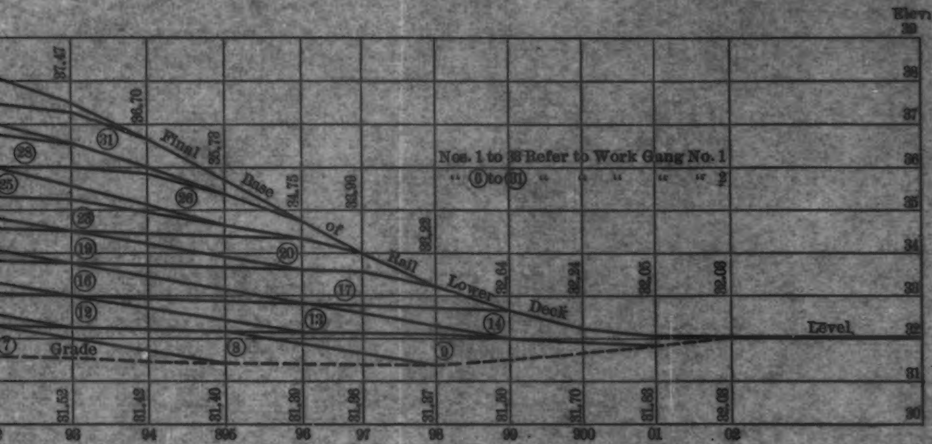
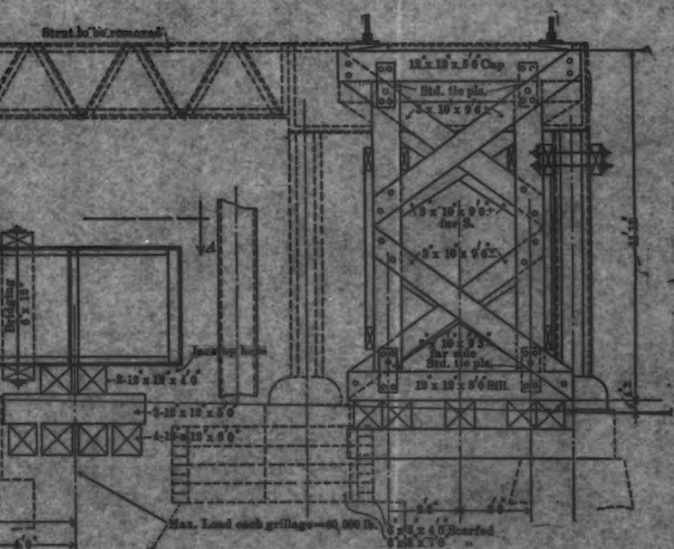




a maximum of about 6 in. Another set of jacks was then started operating, one set continuing north from the location of the fifth operation, and the other set starting at the south end of the grade. The effect of this method of raising the structure was actually to raise it in steps the full length of the rise, each step being approximately 6 in. high. The numbers on the diagram indicate the sequence of the operations, and the numbers enclosed in a circle indicate the operations of the second set of jacks. As this second set was not working during the five first operations of the first set, the first operation of this set is numbered 6, to correspond to the sixth operation of the first set of jacks. It was intended originally to raise the structure between 1 and 5 A. M., when no trains are running on this line, and it was intended to complete one operation only during each night, but, as work of this kind would be very unsatisfactory to perform by artificial light, it was decided to do it during the day, and the jacking operations were completed in 17 working days, without interfering with the operation of trains.

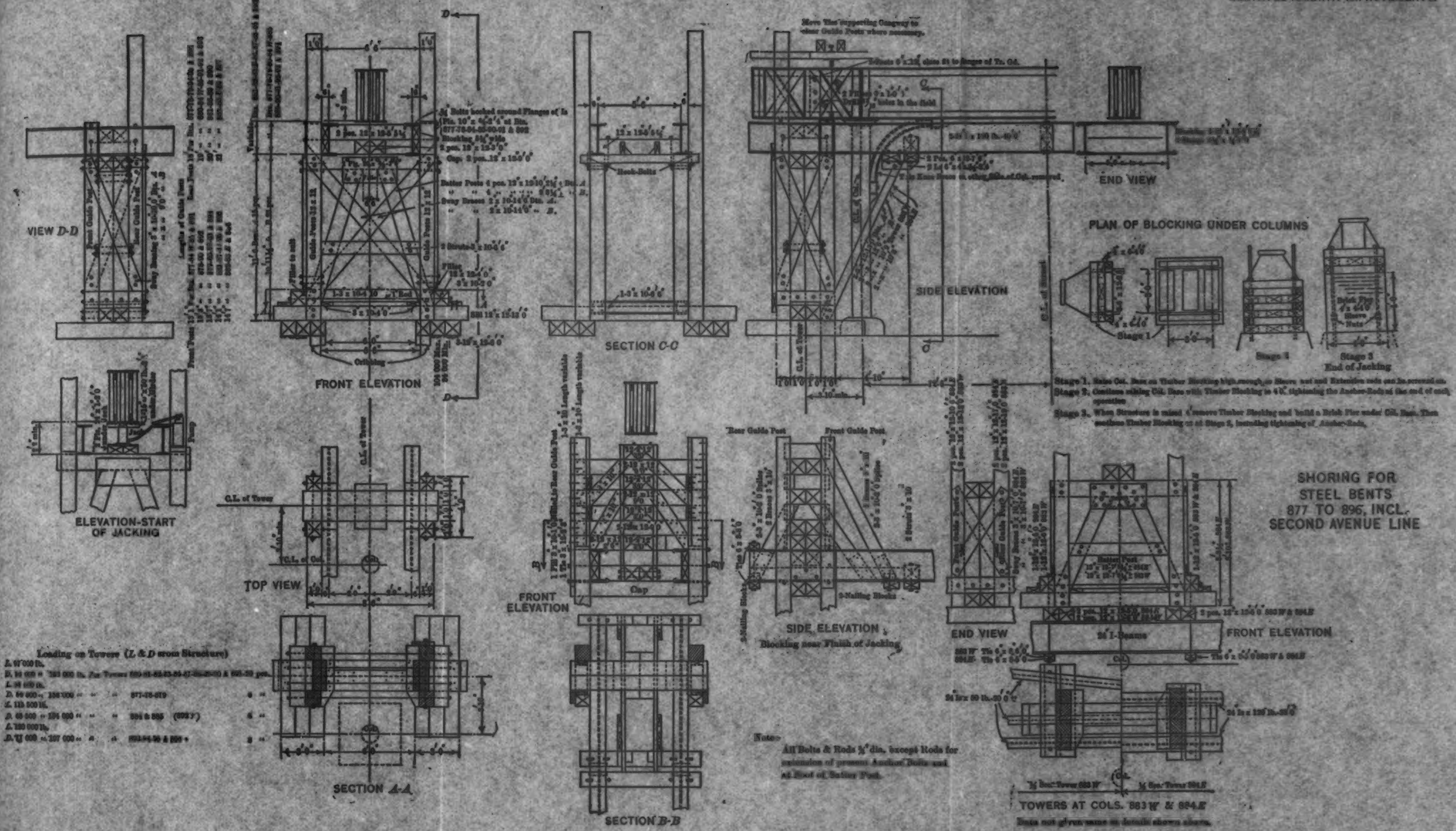
At both ends of the structure raised, where the lift was less than 3 ft. 6 in., that is, where the track level could be raised to its proper grade by raising the track stringers only, without bringing the bottom of the stringers above the top of the cross-girders, the track stringers only were raised. For the remainder of the length, which altogether included twenty bents, the complete structure, including cross-girders and columns, was raised. Where the track stringers only were raised, they were supported during the work by shoring. A four-post bent with all the posts in the same plane and with the two outside posts battered, was set on a grillage of 12 by 12-in. timbers laid on the street surface and grouted. The bent was placed outside of the columns, in order not to interfere with the street-car service, the tracks of which were between the columns. The bent was tied to the column by two yokes. On top of two such bents, at the columns supporting the same cross-girder, were placed four 24-in. I-beams, two on each side of the cross-girder. These I-beams had been used previously for shoring girders during the foundation work on other sections, and, as it was desired not to punch holes in them, in order not to destroy their value for any future use, the I-beams were held in place by wood blocks and steel straps. The jacks used for raising the girders were set on top of these I-beams, and,

during the jacking operation, the space between the lip of the stringers and the top of the cross-girder was continuously kept filled with blocking and wedges, in order to prevent any dropping of the stringers due to failure of the jacks. When a jacking operation was completed, the space between the top of the I-beams and the bottom of the track stringers was also filled with blocking, so as to have double security against dropping of the stringers.

Where the structure was to be raised complete, the method of procedure was as follows: First, a pit was dug around the base of each column, deep enough to expose the base. These pits were 6 ft. square, after being lined solidly with 12-in. timbers. A double A-frame, of 12 by 12-in. timbers, as shown on Plate XV, was then placed under the ends of the cross-girders, outside the columns, on timber blocking laid on the street surface, and grouted. A working platform was provided on top of the A-frames and 12 by 12-in. guide posts at each corner of the platforms. These guide posts were to keep the blocking in place, as will be described later, but the inside ones could not be carried to their full height before the structure was raised, as they would interfere with the track stringers in their original position. They were arranged, therefore, so that they could be spliced afterward. To stiffen the bents laterally during the work, two 24-in. I-beams connected the two shoring bents under the same cross-girder, and were braced by brackets to them.

When all the structure to be raised was shored, the nuts of the anchor-bolts in the bents to be raised during an operation were loosened, and the jacks, which were set on top of the working platforms, were started. The rise of the structure was followed up with wedges and blocking, both under the cross-girder and under the bases of the columns. When the structure was raised high enough to permit it, sleeve nuts were screwed on to the anchor-bolts, and extension anchor-bolts, consisting of 2-in. rods, threaded their full length, were added. Afterward, the structure was secured to the foundations after each jacking operation by tightening the nuts on the extension bolts. On the top of the shoring bents, the working platform was gradually raised by a blocking of 12 by 12-in. timbers secured to the shoring by the guide-posts and by braces to the I-beams, as shown.

Two of the shoring bents had to be placed immediately above public service manhole chambers in the street. They were supported, there-


SHORING FOR
STEEL BENTS
877 TO 896, INCL.
SECOND AVENUE LINE
VIEW D-D
FRONT ELEVATION
SECTION C-C
SIDE ELEVATION
END VIEW
PLAN OF BLOCKING UNDER COLUMNS
Stage 1
Stage 2
Stage 3
End of Jacking
ELEVATION-START
OF JACKING
TOP VIEW
SECTION A-A
FRONT
ELEVATION
SECTION B-B
Rear Guide Post
Front Guide Post
SIDE ELEVATION
Blocking near Finish of Jacking
END VIEW
FRONT ELEVATION
24" I-Beams
Loading on Towers (L & D from Structure)
Note:
All Bolts & Rods 3/4" dia. except Rods for
extension of present Anchor Bolts and
at Root of Batter Post.
TOWERS AT COLS. 883 W & 884 E

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fore, on shoring girders, which removed the load from the roof of the chambers and also permitted the necessary access to them.

Fig. 107 shows a shoring bent during a jacking operation. To raise the track stringers, only 25-ton ratchet jacks were used, one under the end of each pair of stringers. Where the entire structure was raised, 60 and 100-ton hydraulic jacks were used.

With the structure was raised a signal tower and the southerly half of the platforms of the existing station at 127th Street. The track stringers in front of the remainder of the platforms were raised, and, in order to retain them in service, narrow platforms were built on top of the existing ones with steps leading down to the old level.

After the structure was raised to its proper elevation, new columns were placed where the cross-girders had been raised. The old columns were removed, and the new ones were placed by tackle attached to the structure girders. At the same time, the ends of the track stringers which had been raised were remodeled to rest on the cross-girders at their new elevation.

The next operation was the erection of the new station at 125th Street and the construction of the new connections to the Harlem River Bridge. Plate XVI shows the different stages of this work. First, the north-bound traffic was turned into the center track south of the new station and returned to the existing north-bound track at a point just north of the new platforms, so as to serve the platform at the existing station at 127th Street. The deck of the abandoned track was then stripped, and the track stringers were moved sidewise to their new location, except across 125th Street, where the existing girders were removed and replaced with new through bridge girders. Three of these girders were placed to carry the east platform and the new north-bound track. The girders were erected with two gin-poles, one at each end of the girder. The track was then laid, and connection was made so that traffic could run over the new north-bound track. The south-bound traffic was then run over the center track, and the structure changes on the west side of the new 125th Street Station were made. When completed, the south-bound traffic was turned from the center track to the new south-bound track. Work, which during this period had been in progress on the two new island platforms, continued, and the erection of the mezzanine station was started. When sufficiently completed, and connected to the street, the east platform was placed

in service for both north and south-bound traffic, the center track again being used for south-bound traffic past the platform, and the existing station at 127th Street was abandoned and removed.

The outside tracks north of the new 125th Street Station were then moved to their new permanent position, the north-bound under traffic. The moving of the track stringers was done in all cases by sliding them on the top of the cross-girders, the work being done with 25-ton jacks.

The upper-deck structure was erected with a traveler, which was set up at the bottom of the incline just north of the new platforms of the station at 125th Street. This traveler erected all the steel for the upper deck as far as the junction with the Harlem River Bridge. Fig. 109 shows this work in progress. In the first two spans, old center-track stringers were used, with the ends remodeled; the remainder of the track stringers are new plate girders.

Steel Erection.—Section No. 5-C.—This section comprised the new Harlem River Bridge. On account of the necessity of maintaining traffic, the erection of the new bridge had to be carried out in such a manner that traffic over the existing bridge would not be interrupted, and it was decided, therefore, to build a new bridge on a pile platform at a convenient point in the river, so that, when the erection was completed, it could be floated into position. The first question to be settled, therefore, was the site for erecting the bridge. The Company owns a yard with a dock front adjacent to and south of the bridge, on the Manhattan side of the Harlem River. This dock front would have been a convenient location for building the bridge, but would have interfered with the use of the yard, and was also objected to by the War Department, on account of encroachment on the river. Just south of this yard, which was bounded on the south side by 128th Street, there was another dock front, set back of the United States bulkhead line. This, as well as the yard adjacent to it, was leased by the Company, and after obtaining the approval of the War Department, the space between the United States bulkhead line and this dock front was determined on as the site for the erection of the bridge. Fig. 111 shows the property leased and the location of the pile platform.

The bridge was built on a timber platform supported on piles. The platform was arranged so that the two shore spans could be

FIG. 109.—SECOND AVENUE LINE, NORTH OF 125TH STREET.

FIG. 110.—TEMPORARY PILE PLATFORM FOR ERECTION OF BRIDGE SPANS.

PLATE XVI.
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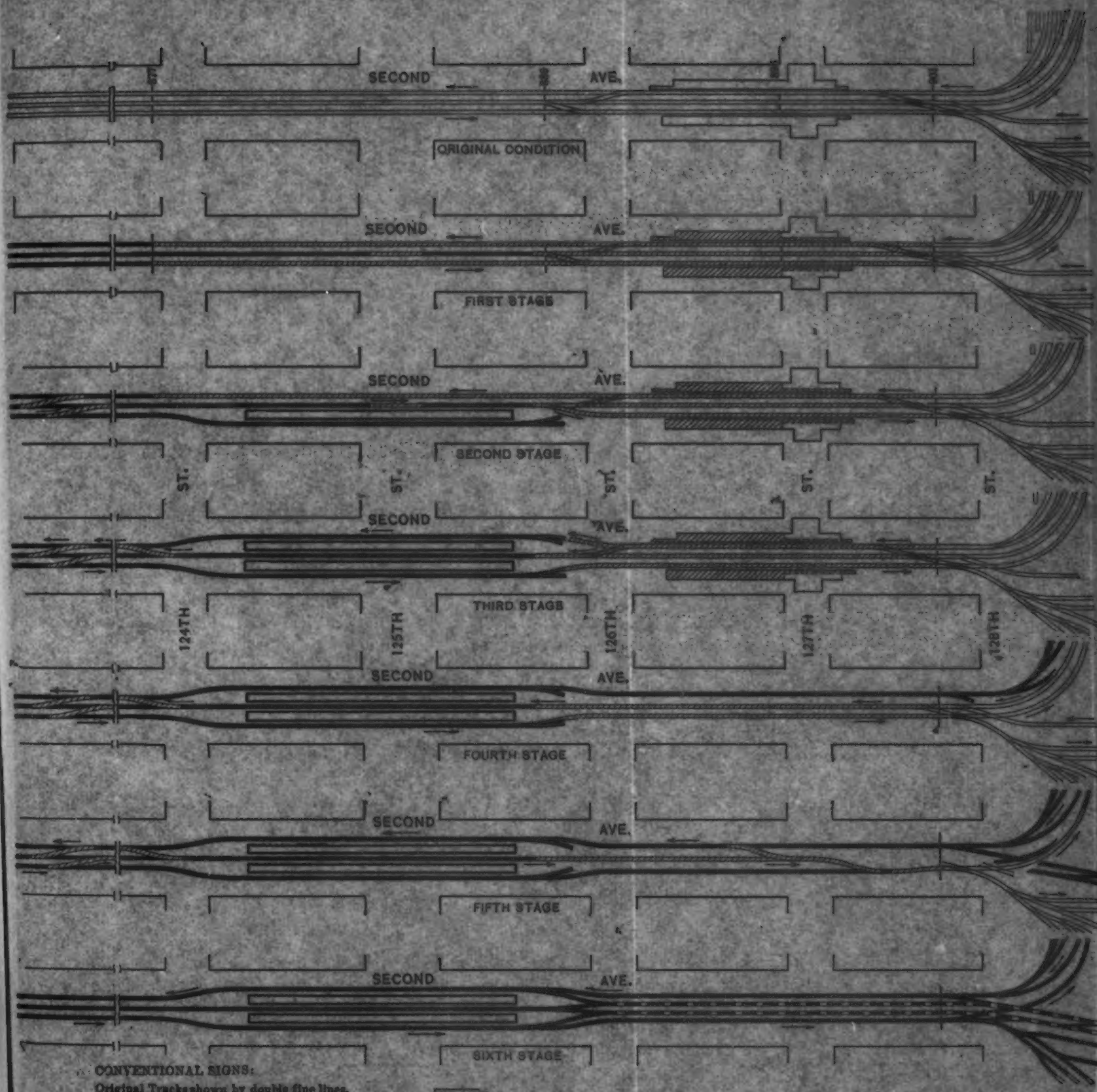
ELEVATED RAILWAY IMPROVEMENTS.



FIRST STAGE. Jack up Longitudinal Girders and Track to Permanent New Grade of Local Tracks from Bent 879 to Bent 901; also, Cross-Girders from Bent 878 to 896. Jack up Platforms with Structure from Bent 896 South. Construct Narrow Temporary Platforms on Old Platforms, to the New Grade of Tracks from Bent 888 North. Erect New Street Columns from Bent 877 to 894 and Sidewalk Columns for New Station at 125th St. and lengthen out Cross-Girders for Station. Make Alterations in Old Longitudinal Girders from Bent 870 to Bent 876.

Remove Platform from 125th St. to Harlem River Bridge. Construct New Foundations at Harlem River. Make all necessary alterations in Old Structure to this point. This work may overlap next Three Stages.

SECOND STAGE. Put in Cross-Overs South of 125th Street, as shown. Turn North-Bound Traffic on to Center Track from 124th Street to 128th Street. Remove Old North-Bound Track over this Stretch. Move Old North-Bound Longitudinal Girders into new locations from Bent 878 to 888. Erect New Longitudinal Girders for East side from 877 to 890. Erect East Platform Girders and Build East Platform of 125th Street Station. Build East half of Station. Lay New North-Bound Track up to and including Switch South of 120th Street.

THIRD STAGE. Put in Cross-Over South of 125th Street, as shown. Connect up Old and New North-Bound Track at 125th Street. Turn South-Bound Traffic on to Center Track form 128th Street to 124th Street. Turn North-Bound Traffic on to New East Track. Repeat operations of 2d Stage for West Track and West side of Structure.

Remove Switch at 128th Street. Connect up old South-Bound with New South-Bound at this point. Turn Traffic over this route. Remove Old Longitudinal Girders for Center Track at 123th Street Span. Erect New Floor-Beams and Stringers of Center Track and complete Station.

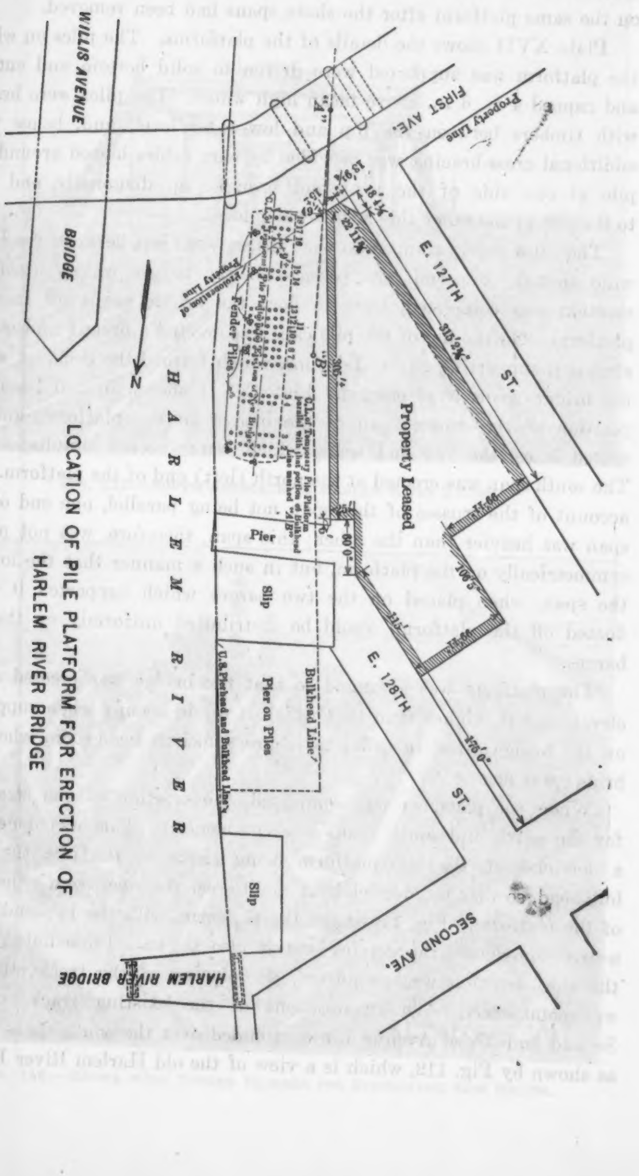
FOURTH STAGE. Move to New Station. Turn South-Bound Traffic on to Center Track past Old Station and past New Station. Tear out Old Station and Platforms. Erect New Longitudinal Girders for West side from Bent 890 to Bent 904. Move out Old Longitudinal Girders with track from Bent 893 to 902 and make new Connections with West Yard.

FIFTH STAGE. Turn South-Bound Traffic on to New South-Bound Track. Erect New Longitudinal Girders for East side from Bent 890 to Bent 902. Move out Old Longitudinal Girders with Track from Bent 891 to Bent 902. Turn North-Bound Traffic on to Center Track. Make temporary Cross-Over between East and West Tracks at 127th Street. Run all Bronx Traffic over Single East Track North of 129th Street. Remove Old Center Track from 127th Street around Curve. Lay New North and South-Bound Bronx Tracks for Lower Deck, as far as possible.

SIXTH STAGE. Complete New East Track between 127th Street and Harlem River Bridge. Turn North-Bound Traffic on to East Track. Remove Center Track from 128th Street to 127th Street. Connect South-Bound Track with South Bound over Harlem River Bridge. Complete North-Bound Local Track from 127th Street North. Make Connection with and complete changes in East Yard. Erect and complete Upper Deck Structure.

CONVENTIONAL SIGNS:
Original Tracks shown by double fine lines.
Final Tracks (lower deck) shown by heavy full lines.
Final Tracks (upper deck) shown by heavy dash lines.
Temporary Tracks shown by hatched double fine lines.
Original Station Platforms shown by fine line outlines.
Final Station Platforms shown by heavy line outlines.
Temporary Station Platforms shown by hatched outlines.
Arrows show direction of traffic movement.

DIAGRAM OF PROGRESS OF ERECTION
SECOND AVENUE LINE



LOCATION OF PILE PLATFORM FOR ERECTION OF HARLEM RIVER BRIDGE

FIG. 111.

erected at the same time, and so that the swing span could be erected on the same platform after the shore spans had been removed.

Plate XVII shows the details of the platforms. The piles on which the platform was supported were driven to solid bottom and cut off and capped 4 ft. 6 in. above mean high water. The piles were braced with timbers between the top and low-water level, and, below this, additional cross-bracing was provided by wire cables looped around the pile at one side of the bent and brought up diagonally and tied to the pile at the other side at low-water level.

The piles were arranged so that spaces were left between the bents wide enough to permit the passage of the barges which, when the erection was completed, were to float the bridge spans off the pile platform. On the top of the pile caps was erected a braced timber construction supporting 24-in. I-beams, which formed the deck, on which the bridge spans were erected. Plate XVII shows, in full lines, the position of the center span of the bridge on the platform, and, in dotted lines, the two end spans which were erected simultaneously. The south span was erected at the north (left) end of the platform. On account of the trusses of this span not being parallel, one end of the span was heavier than the other; this span, therefore, was not placed symmetrically on the platform, but in such a manner that the load of the span, when placed on the two barges which supported it when floated off the platform, would be distributed uniformly on the two barges.

The platform was arranged so that the bridge was erected at an elevation 2 ft. higher than that which it would occupy when supported on the bridge piers, in order to insure sufficient head-room when the bridge was floated in.

When the platform was completed, the erection of the steelwork for the north and south spans was commenced. This was done with a derrick-boat, the pile platform being about 40 ft. from the river bulkhead, so that the derrick-boat could erect the steel from either side of the platform. Fig. 110 shows the platform, with the two end spans nearly completed; the derrick-boat is also shown. Immediately after the steel erection was completed, the placing of the track structure was commenced. The connections of the existing tracks of the Second and Third Avenue Lines extended over the south shore spans, as shown by Fig. 112, which is a view of the old Harlem River Bridge.

FIG. 112.—SOUTH APPROACH TO HARLEM RIVER BRIDGE.

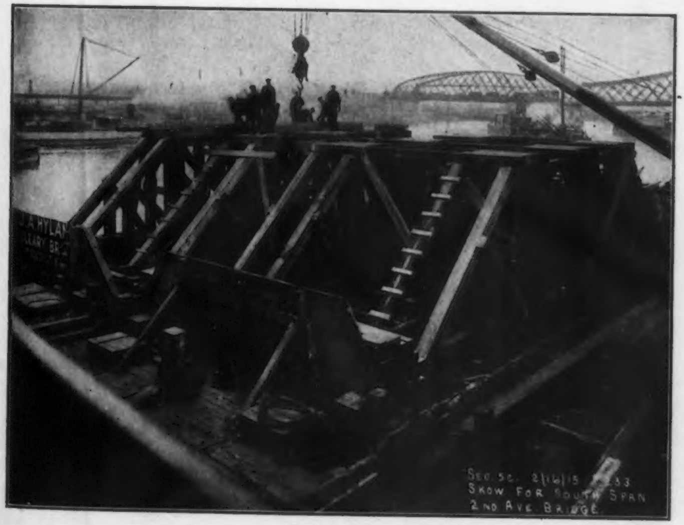


FIG. 113.—SCOWS WITH TIMBER TRUSSES FOR SUPPORTING NEW BRIDGE.

PLATE XVII.
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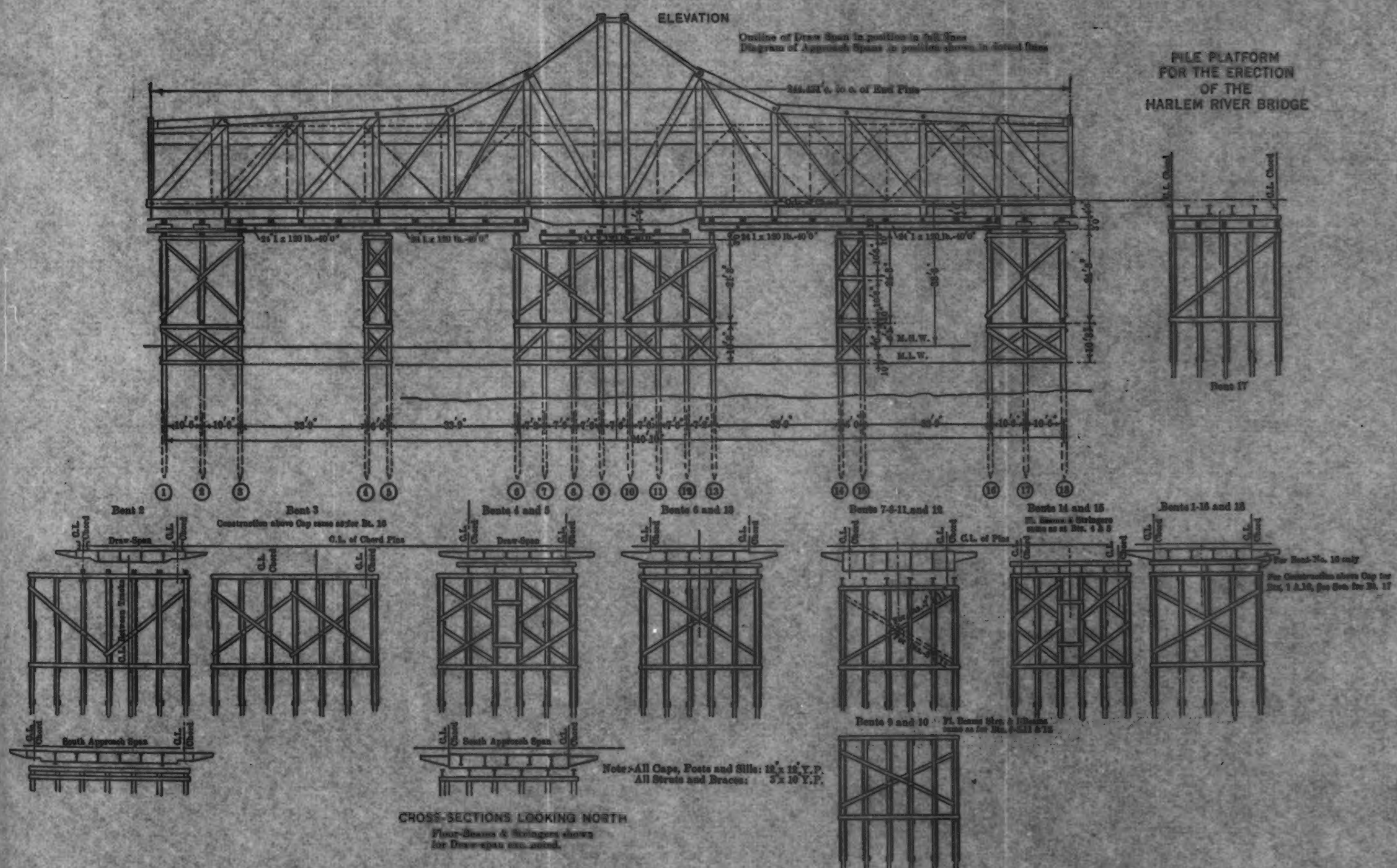
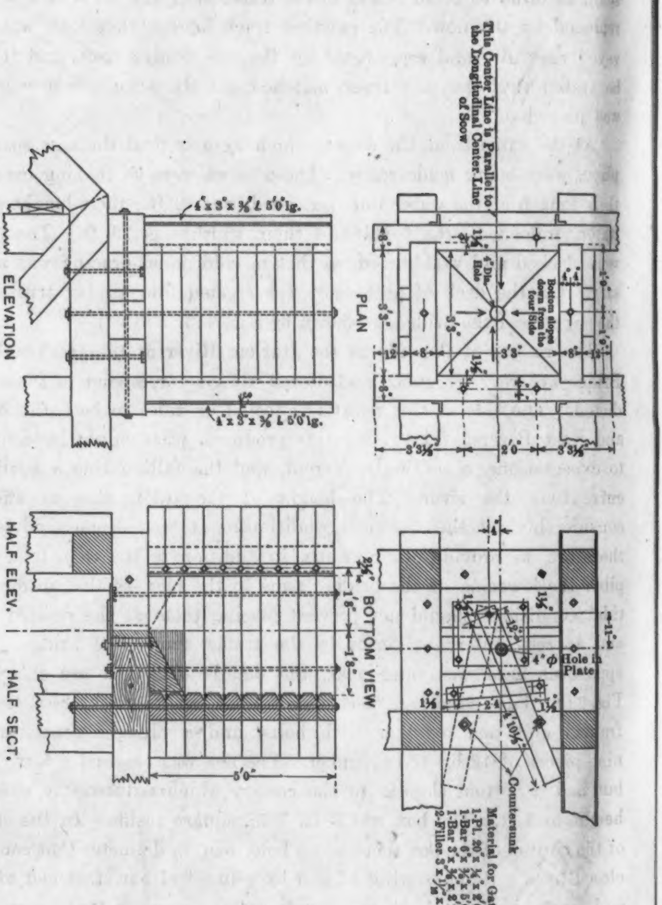


It was necessary that the alignment of the track laid on the new span should be an exact duplicate of that of the track existing on the old span, in order to avoid delays to the traffic after the old span had been replaced by the new. The existing track lay-out therefore, was surveyed carefully and reproduced on the new bridge span, and it may be stated that the new track matched exactly when the new bridge was placed.

At the same time, the scows, which were to float the new spans in place, were being made ready. These scows were 95 ft. long over all; their length at the water line, unloaded, was 80 ft.; their height above water, unloaded, was 6 ft., and their width was 29 ft. The scows were decked and well-braced, so that no additional bracing was necessary. On the deck of each scow was erected four timber trusses for the support of the bridge, as shown by Fig. 113.

The ranges of the tide in the Harlem River at the Harlem River Bridge are very irregular, and do not always correspond to those predicted. The tide at this point is affected by tides in both the North and East Rivers. The rising tide produces, what might be contrary to expectations, a southerly current, and the falling tide a northerly current in the river. The height of the tides, also, is affected considerably by the weather conditions. It was deemed advisable, therefore, to provide other means, in addition to the tide, to accomplish the lowering of the bridge spans to the pier, so that unforeseen tidal conditions should not prevent placing them at the desired time and thereby interrupt seriously the traffic across the bridge. The apparatus used was sand-jacks, the details of which are shown by Fig. 114. Each sand-jack consisted of a box, made of 6 by 8-in. timber, framed and tied together with bolts, and a plunger consisting of nine pieces of 12 by 12-in. timber. The box was originally 5 ft. high, but had a bottom sloping to the center, which reduced the effective height to 4 ft. The box was 3 ft. 3 in. square inside. In the center of the bottom of the box there was a hole, 4 in. in diameter that could be closed by a gate consisting of a 6 by $\frac{3}{8}$-in. steel bar, fastened at one end to the under side of the box in such a manner that by moving the other end of the bar sidewise, the opening could be closed or opened as desired. Six of these boxes were set on top of the timber trusses on each scow and bolted securely to the framework of the trusses. To the 24-in. I-beams, supporting the bridge spans on the



SAND BOX
FIG. 114.

timber platform, were bolted plungers, as shown by Fig. 115, in such positions that they would fit accurately over the boxes on the scows, when the latter were in their proper places, ready to lift the bridge spans off the platform. The plungers consisted of nine pieces of 12 by 12-in. timber, set on end and bolted together so that their outside dimensions were 3 ft., leaving a clearance of 1½ in. all around between the plungers and the boxes. The method of using the sand-jacks was to fill the boxes to the top with fine dry sand and let the plungers, which carried the bridge spans, be supported on top of the sand. When it was desired to lower the bridge, the gates at the bottom of the sand boxes were opened, and the sand flowed out, letting the plungers descend gradually in the boxes, and with them the bridge.

The south span of the bridge was first made ready to be moved. The time for the replacement was determined in conjunction with the Company's Traffic Department, which desired this to be the night between a Saturday and a Sunday, after the hour of 1 A. M., at which time the traffic over the bridge was lightest. As the rising tide was to be used for removing the old span, it was necessary to select a night when high tide occurred an hour or two after 1 P. M. After consulting the tide tables, Sunday morning, February 20th, 1915, was selected as the time for replacing the south span. As it was not deemed advisable to keep the span floating on the scows longer than necessary, on account of the possible danger of scows leaking and settling, or even sinking, it was decided to float the bridge span off the platform on the tide immediately preceding the one on which the span was to be set in place. This made the time for floating off the span, Saturday, February 19th, in the afternoon.

At 8 A. M. on this date, the two scows which were to carry the span were brought into their proper place, as shown by Fig. 115. The floating weight of the south span was 380 tons, which would give the scows a displacement of 2 ft. 8 in. It happened at this time that the tide did not rise more than 3 ft., and as about 4 in. was lost before the plungers got proper bearing on the sand, the scows failed to float the bridge clear of the platform. The replacing was postponed for 24 hours, which was satisfactory to the Traffic Department, as February 22d was a holiday, and the traffic, therefore, light, and

the time of high water would not change appreciably during the 24 hours.

The scows, which had been removed from under the bridge span to avoid the danger of unexpectedly floating the bridge during the intervening high tide, were again placed in position on Sunday morning, February 21st. As a steady west wind had produced exceptional low ranges of tide, the sand boxes were increased 12 in. in height, and jacks were used between the bridge span and the scows to produce an initial displacement of the scows of 8 in. at low tide. The south span was floated successfully at this time, 2 hours before high tide, with 9 in. of tide to spare, although the range of tide was only 2 ft. 11 in. When the span was clear of the platform, the base of rail was 36 ft. 7 in. above the water level at the south end of the span, and 35 ft. 10 in. at the north end. The displacement of the scow at the south end was 2 ft. 6 in. and at the north end 2 ft. 10 in.

As the scows gradually cleared the platform, they were braced together by timbers cut in advance for this purpose, so as to insure the scows remaining in their relative position during the towing. The bridge was towed to the Company's dock and was tied up there to await the next tide, when it was to be placed.

In the mean time two other scows had been provided with timber trusses similar to those on the scows carrying the new bridge span, and, at 10 P. M., when the next low tide occurred, they were towed to and placed under the old south bridge span. Blocking was placed between the timber trusses and the under side of the old bridge, and at 1.30 A. M., the blocking was permitted to bear. The last train passed over the bridge at 1.58 A. M., and all the rail connections between the bridge span and the structure on the shore were removed by 2.07 A. M. At the same time, the swing span was opened so as not to interfere with the work.

At 3.30 A. M., the old span was clear of the supporting piers, except at the northeast corner, and as the tide was approaching its highest level, the derrick-boat, which had been used for erecting the bridge, was used to assist the tide in getting the span clear of the piers at this point. The old span floated clear at 3.58 A. M. on top of the tide, and was towed to the north and afterward returned through the open swing span to the Company's dock.

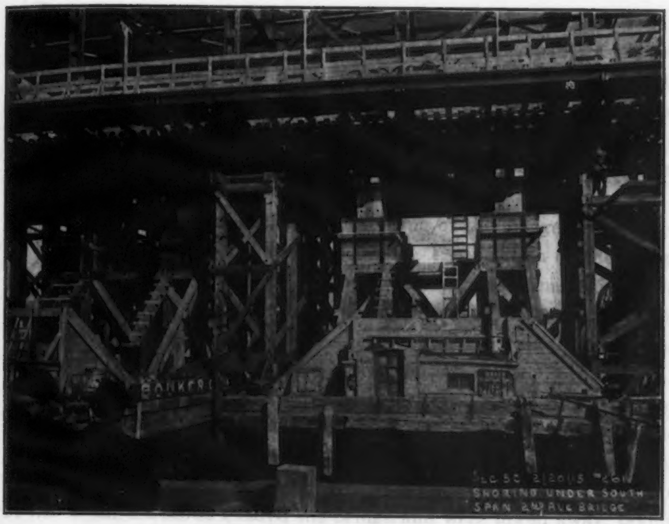


Fig. 115.—Sand Jacks.

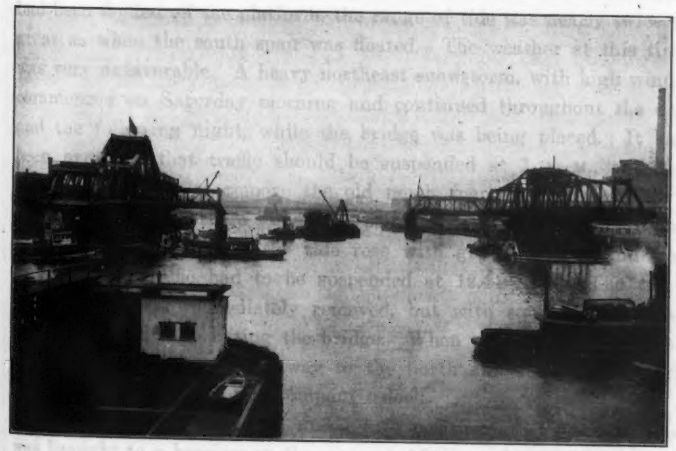


Fig. 116.—Harlem River Bridge During the Changing of the Swing Span.

Immediately thereafter, the new bridge seats and expansion rollers were set on the piers by small derricks, and the new span was floated into position by 4.30 A. M., 3 ft. 6 in. above its proper elevation. The tugboats maneuvered the bridge very closely to its proper location, and it was then secured to the piers by two crossed sets of blocks and falls at each end and by steamboat ratchets to control longitudinal movements.

The tide was now falling, and the settling of the bridge was assisted by letting sand flow out of the sand-jacks. The position of the bridge span was kept in continuous adjustment, and a careful final adjustment was made immediately before the bridge was about to bear on its seats, by inserting a bar between the ropes of the falls and twisting the ropes until the bridge was accurately centered.

The new south span was brought to a bearing at 6.10 A. M. The track connections were then made, and the swing span closed, and at 6.56 A. M., 4 hours and 58 min. after traffic had been suspended, the first train passed over the new bridge. The tide continued to fall, and at 7.40 A. M., the scows cleared the bridge and were towed away, taking all the falsework with them.

The next favorable tide for setting a bridge span occurred on Sunday morning, March 6th, 1915. The day before the new north span had been floated off the platform, the range of tide was nearly twice as great as when the south span was floated. The weather at this time was very unfavorable. A heavy northeast snowstorm, with high winds, commenced on Saturday morning and continued throughout the day and the following night, while the bridge was being placed. It had been arranged that traffic should be suspended at 1 A. M., and the scows which were to remove the old north span were brought to a bearing under the bridge some time prior to this. On account of the northeast wind, however, the tide rose with great rapidity and was so high that traffic had to be suspended at 12.44 A. M. The track connections were immediately removed, but with some difficulty, as the tide was already lifting the bridge. When clear of the piers, the old north span was towed away to the north and returned through the open swing span to the Company's dock.

The new north span was towed into position at 12.35 A. M. and was brought to a bearing on the piers at 4.31 A. M. by using the sand-jacks, while the tide was still 1.0 ft. above mean high water.

The erection of the new swing span commenced on March 10th, 1915, and the placing of the new operating machinery was started on April 5th. As described previously under the heading "Details of Design", the machinery first erected did not work properly, on account of the flexibility in the support of the bearings of the beveled gears. New castings, with bearings for both the gear wheels made in the same casting, therefore, were designed and fabricated, and the erection of the machinery was not finally completed until August 15th, 1915.

In the mean time the necessary preparations on the existing center pier for carrying the new swing span had been in progress. As stated, the new swing span was center-bearing, and the old one was drum-bearing. In order to provide space for the center casting, it was necessary to excavate a pit in the center of the pier, 8 ft. in diameter and 4 ft. 3 in. deep, directly under the center pivot of the old swing span. A frame of I-beams, therefore, was placed on top of the pier to support the pivot, and was held in position securely by timber struts wedged in between the steel frame and recesses in the pier made for the purpose. The excavation was then started, by what amounted to sinking a shaft into the pier outside of the center pivot, and, from that, the pit for the new center casting was tunneled out. The work was greatly hampered by lack of head-room, having to be done, until sufficient depth had been reached, while lying on top of the pier. When the excavation was completed the bottom of the pit was made level with a granolithic finish, and a beveled ring with an inside diameter of 7 ft. was set in place and anchored securely to the pier. In order to insure a uniform bearing, a sheet of lead, $\frac{1}{16}$ in. thick, was placed under the beveled ring. The purpose of this ring was to act as a guide when the center casting was to be set in place, which was to be done simultaneously with the placing of the swing span, so that no time should be lost in instrument work while the traffic was suspended.

In addition to this work, plates were placed on the pier and anchored and grouted to form seats for the new wedge castings, and the rack for the new bridge, which was made in twelve sections, was placed in position and anchored inside of the existing circular track. This track, which had a tapered surface, was maintained temporarily to act as a

track for the balancing wheels of the new bridge, but was renewed after the latter had been put in place.

On the rest piers, the existing granite cap-stones were cut to fit the new wedge and latch castings. As the new wedge castings were at the same points as the existing castings, it was necessary to remove these while the seats were being prepared. This was done between 5 P. M. and 9 A. M., during which time the bridge is not opened for river traffic.

Before the new swing span was moved, the lower-deck track had been laid complete, and a temporary switch machine had been placed on the new bridge. A new submarine cable was also laid to the center pier to supply power for operating the new swing bridge.

The day chosen for placing the swing span was Sunday, August 22d, 1915. At 12.35 P. M., on August 21st, the four scows which were to carry the bridge were placed in position under the new swing span on the platform. The plungers came to a bearing on the sand in the boxes at 1.15 P. M. At 5.40 P. M., the tide had lifted the bridge clear of the platform 2 hours before and 2 ft. below high tide. The floating weight of the swing span was 1 100 tons, and the average displacement of the four scows under this load was 3 ft. 6 in. Fig. 117 shows the four scows taking the load of the swing span off the platform. The base of rail, when floating, was 35 ft. 2 in. above water level. The bridge was towed over to the east side of the river, just south of the Harlem River Bridge, and was tied up there until the time of placing it had arrived. On Sunday morning, August 22d, low tide occurred at 1.30 A. M. The two scows which were to float off the old span on the rising tide were placed under that span and were allowed to take bearing at once. Traffic across the bridge was suspended at 1.54 A. M. At 5.30 A. M., the old span was lifted clear of the piers and towed to the south and tied up at the Company's dock. The old spider casting and bearing wheels were left on the center pier when the old span was removed. After cutting the spider in convenient pieces to be handled, all these parts were removed by a derrick-boat, which also set the new center casting and the center pier wedges. At the same time, the latch castings and the wedge castings on the end piers were set by small derricks placed on the upper deck of the end spans. Fig. 116 shows the derrick working at the center pier, and also shows the old center span removed to the right and the new span waiting to be placed at the left.

The new span was floated into its proper position over the piers at 7.45 A. M., and was secured to the end spans in proper alignment in a manner similar to that described for the south span. In addition to this, 6 by 8-in. timbers were bolted to the approach spans and acted as guides for the end post of the swing span. The lowering of the bridge was then commenced by the sand-jacks, aided by the falling tide, and, at 10 A. M., the swing span landed in its final position.

The falling tide released the scows at 11.30 A. M., and at 12.43 P. M., when the track and signal work was completed, the first train crossed the new span.

Steel Erection.—Section No. 5-D.—This section extended from the Harlem River through the yard of the New York, New Haven and Hartford Railroad and the Company's yard to 133d Street. Steel erection had been in progress north of this section, and the traveler doing this work proceeded directly from the north to this section at 133d Street. In order to place the new structure across 133d Street, the structure had to be shored as some of the supporting columns were moved. Generally speaking, one of the two columns which supported the structure carrying one of the tracks was removed, and the structure was supported temporarily by an A-frame, as shown by Fig. 118, braced diagonally to the remaining column. South of 133d Street the tracks were carried directly above a machine shop in the Company's yard. In order to place the supporting columns, which went through the roof and the floor of the shop, and were supported on foundations below the shop floor, the roof was entirely removed. After the structure was erected, the roof was replaced.

The traveler proceeded through the Company's yard and the freight yard of the New York, New Haven and Hartford Railroad, the steel being furnished on flat cars operating on the existing tracks of the elevated structure.

Steel Erection.—Section No. 6-A.—Prior to erecting the new steelwork in this section, which comprises the Third Avenue Elevated Line, from 133d to 147th Streets, through the Company's private right of way, the existing structure had to be moved to make room for the new one. The existing tracks through the right of way were supported on brick piers with granite cap-stones. New concrete piers were built to support the south-bound track at its new location. The north-bound track, including the track stringers, was shifted 6 in. to the east. The

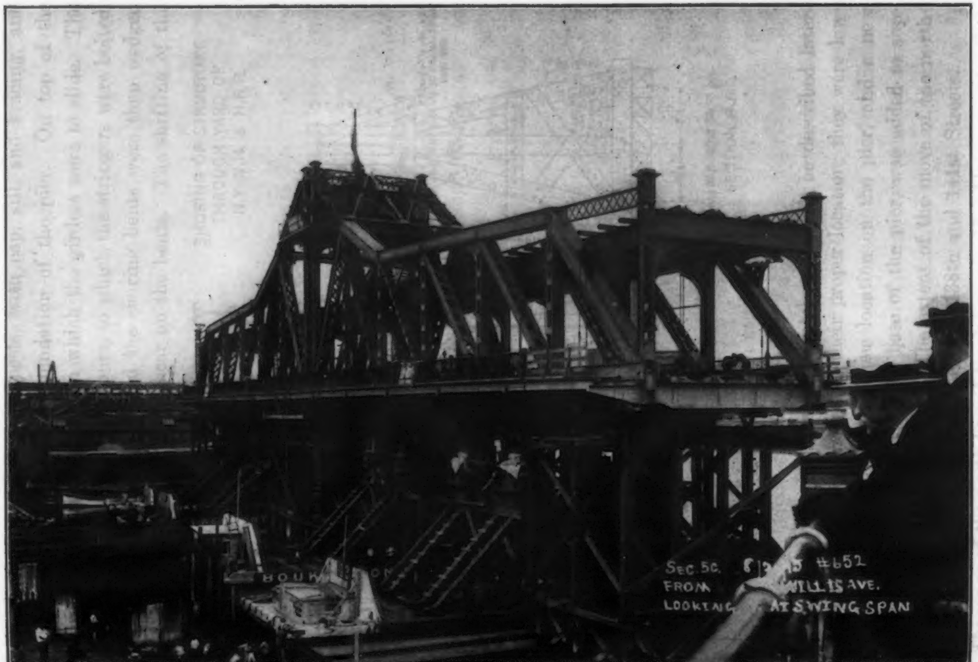


FIG. 117.—SCOWS READY TO LIFT SWING SPAN.

work was done by supporting the track stringers on wooden bents placed close to the piers, one on each side of the pier. The bents consisted of two 12 by 12-in. posts, with cap, sill, and bracing, and with the sill bearing on the foundation of the pier. On top of the bents were placed steel plates on which the girders were to slide. The stringers and the granite cap-stones, to which the stringers were bolted, were jacked up about 1 in., and the shoring bents were then wedged up so as to make the stringers bear on the bents. The shifting of the

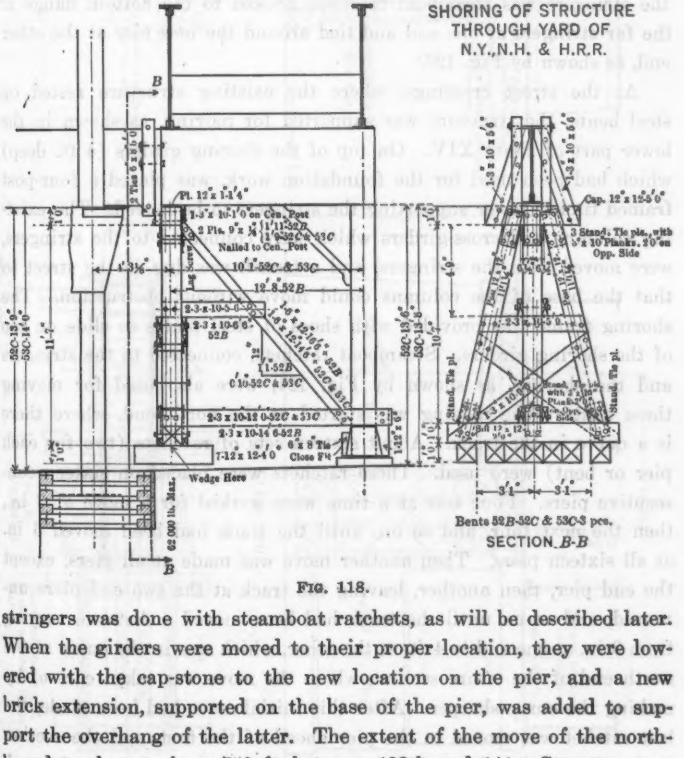


FIG. 118.

stringers was done with steamboat ratchets, as will be described later. When the girders were moved to their proper location, they were lowered with the cap-stone to the new location on the pier, and a new brick extension, supported on the base of the pier, was added to support the overhang of the latter. The extent of the move of the north-bound track was about 750 ft. between 138th and 141st Streets.

The move of the south-bound track was considerably more extensive, both in length and distance. The track was moved between 133d and 144th Streets, a distance of 2 760 ft., and the sidewise movement was

from 12 to 13 ft. The track stringers were shifted from their existing piers to the new piers, which were built in advance of this work, by sliding them over a track, as shown by Fig. 119. Between the piers, and on top of the new pier, the track consisted of rails, supported on the pier on 6 by 8-in. ties and between the piers on timber bents or blocking, as shown. On the old piers, plates ¾ in. thick were inserted between the stringers and the top of the piers, on which the stringers were to slide during their moving. The apparatus used for shifting the stringers was steamboat ratchets, hooked to the bottom flange of the far stringers at one end and tied around the new pier at the other end, as shown by Fig. 120.

At the street crossings, where the existing structure rested on steel bents, the structure was supported for moving, as shown in the lower part of Plate XIV. On top of the shoring girders (4 ft. deep) which had been used for the foundation work, was placed a four-post framed timber tower supporting the stringers to be moved. The existing columns and cross-girders which were connected to the stringers, were moved with the stringers, and a trench was dug in the street so that the base of the columns could move without obstruction. The shoring frame was provided with shoes of steel plates to slide on top of the shoring girders. Steamboat ratchets, connected to the stringers and the shoring, as shown by Fig. 121, were also used for moving these bents. The moving was started at the north end, where there is a curve in the track. About sixteen sets of ratchets (two for each pier or bent) were used. These ratchets were placed on sixteen consecutive piers. Four sets at a time were worked for a move of 3 in., then the next four, and so on, until the track had been moved 3 in. at all sixteen piers. Then another move was made at all piers, except the end pier, then another, leaving the track at the two end piers unmoved, and so on until the track had been moved a distance varying from 3 in. at one end to 4 ft. at the other, which was in the curve at the north end of the entire section, where the move was taken care of by making the curve sharper. After this initial move had been made, the last ratchet was moved to the pier ahead of the first, and the section in hand was moved another 3 in., then another ratchet was moved, and another 3-in. move was made, and so on until the entire track was moved 4 ft., except at the southerly sixteen piers, where the length of the move varied down to 3 in.

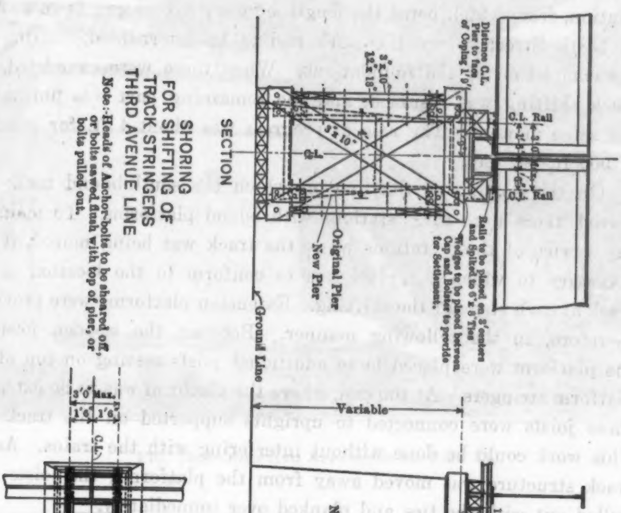

SECTION
ELEVATION
PLAN
SCHEME 3
SHORING
FOR SHIFTING OF
TRACK STRINGERS
THIRD AVENUE LINE
Note:-Heads of Anchor-bolts to be sheared off, or bolts sawed flush with top of pier, or bolts pulled out.
Ground Line
Variable
Original Pier
New Pier
Proposed position of Girders
Present position of Girders
Blocking to Support end of plates
12' Max.
3'0" Max.
C.L. Rail
C.L.
New Pier
Original Pier

FIG. 119.

Then, two more 4-ft. moves were made in the north end, in a similar manner, bringing the track in final position past the 143d Street Station, from which point the length of the move varied down to 3 in. at 138th Street, where the work had to be interrupted, waiting for the completion of the foundations. When these were completed, the track shifting was resumed, and the remaining part was finished in the same manner. The rate of progress was about 4 ft. for a length of 900 ft. per day.

On the portion of the line on which the south-bound track was moved, there were three stations with island platforms. To maintain the service of these stations while the track was being moved, it was necessary to widen the platforms to conform to the location of the track at each stage of the shifting. Extension platforms were provided, therefore, in the following manner. Between the wooden joists of the platform were placed loose additional joists resting on top of the platform stringers. At the end, where the platform was to be extended, these joists were connected to uprights supported on the track ties. This work could be done without interfering with the trains. As the track structure was moved away from the platforms, the joists were pulled out with the ties and planked over immediately.

The length of the track in its new position was about 1 ft. less than in its original position. It was necessary, therefore, to offset the ends of the track stringers in two adjacent spans at a number of places, so as to permit the girders to slide past each other. To take up the shortening of the rails, two expansion rail joints were introduced in the track.

The steel for the new structure was erected with a traveler, starting at the north end of the section. The material was usually trucked to the intersecting streets, where it was unloaded and moved on rollers into the right of way, where the traveler picked it up.

Steel Erection.—Section No. 6-C.—The erection work on this section, which included the Third Avenue Line from 147th Street to Fordham Road, comprised generally the replacement of existing columns with new ones, the erection of center-track stringers, and the remodeling of the structure at the stations.

Altogether, 699 columns were replaced on this section. While the work was being done, the cross-girders were shored, as shown by Fig. 122. Prior to placing the shoring, the rivets connecting the columns to

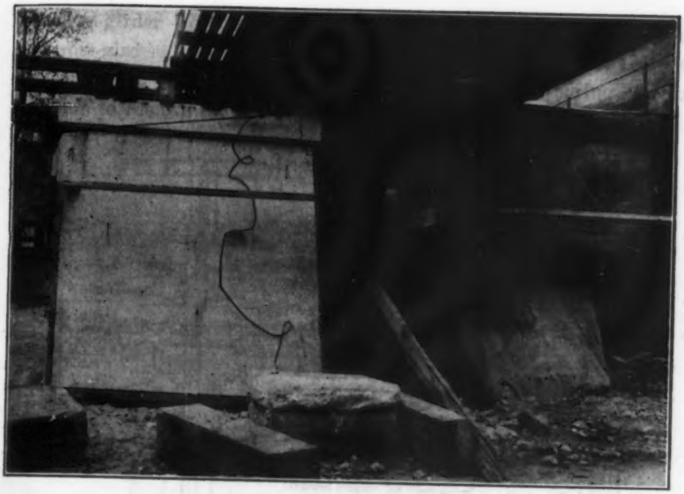
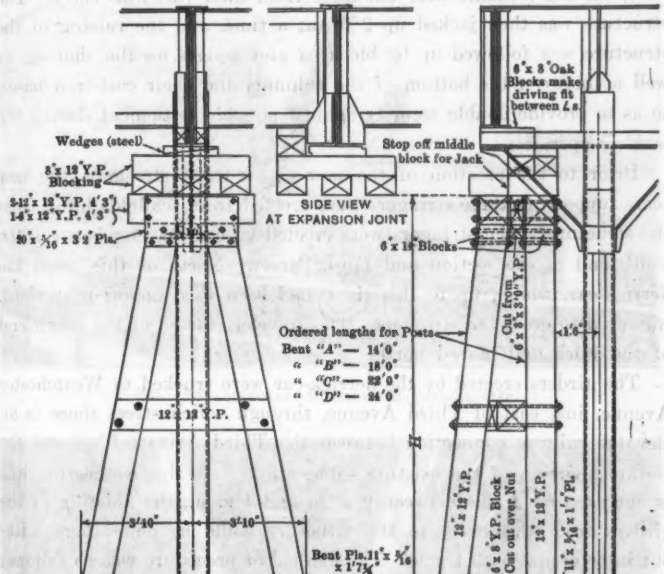


FIG. 120.—SHIFTING APPARATUS ON PIER 75.

FIG. 121.—SHIFTING APPARATUS ON BENT 73.

the cross-girder were cut out, and new stiffeners were provided on the cross-girder. The shoring was then placed, and the cross-girder was jacked up sufficiently to transfer the load to the shoring and to get room to remove the old column and place the new one.

SHORING OF CROSS-GIRDERS



SIDE VIEW AT TYPICAL CROSS-GIRDER

END VIEW OF BENT

FIG. 122.

In conjunction with the placing of the new columns, the work of raising the entire structure was carried out at stations, where the new arrangement provided a mezzanine station below the track structure. Generally, the head-room under the structure was not sufficient for the mezzanine, and, in some cases, the structure had to be raised 19 in. The work was done by first placing the shoring previously described for replacing the columns under all the cross-girders to be

raised at a station; then the rivets in the end-connection angles of the track stringers were cut out and replaced by bolts, and an additional bearing for the stringers was provided by wedging oak blocks between the bottom flanges of the stringers and the cross-girders. At the same time, the old columns were cut loose from their cast-iron bases. The structure was then jacked up 2 in. at a time, and the raising of the structure was followed up by blocking and wedges on the shoring, as well as between the bottom of the columns and their cast-iron bases, so as to provide double security against possible settlement during the jacking operations.

Prior to the erection of the new center-track stringers, the new seats supporting these stringers were erected from scaffolds hung from the structure. The stringers were erected with a traveler between the south end of the section and 176th Street. North of this point the derrick-car, which, up to this time, had been used on other sections, was used to erect the stringers. The traveler started at the south end of the work and moved north.

The girders erected by the derrick-car were trucked to Westchester Avenue, just east of Third Avenue, through which street there is an elevated railway connection between the Third Avenue Line and the elevated portion of the existing subway line. As this connecting line is not used for traffic between 9 A. M. and 4 P. M., the hoisting of the girders from the street to the structure could be done there without interference with traffic. The method of procedure was as follows: The train, consisting of the derrick-car, a flatcar, and a motor-car, was run in on this connecting branch, and two girders were hoisted and placed on the flatcar. The train then proceeded north on the Third Avenue Line immediately behind a passenger train. When it arrived at the point where the girders were to be erected, the derrick-car was blocked and guyed, and one of the girders was erected. If there was time to erect the other girder without holding up the next train behind, it was done, otherwise the derrick-car train was started after erecting the first girder, proceeded to a cross-over, returned on the south-bound track and erected the second girder. In either case, the train returned on the south-bound track to the branch in Westchester Avenue for a new load. Fig. 123 shows the hoisting of the stringers at Westchester Avenue. It was found that the cost of erecting the steel with the traveler was nearly 50% higher than with the

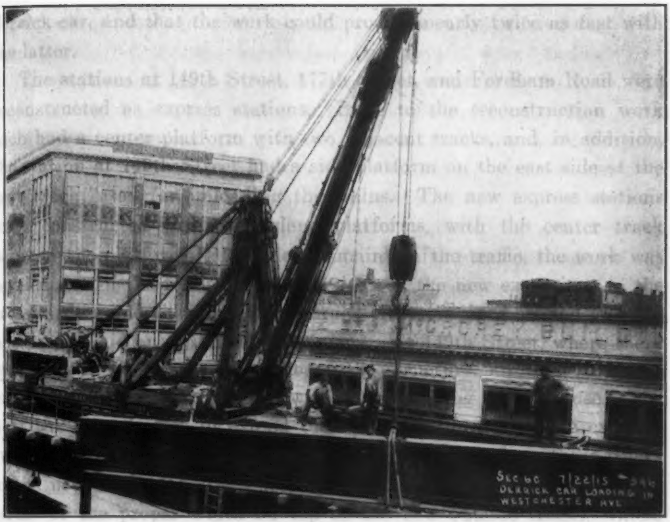


FIG. 123.—ERECTING STEEL WITH DERRICK CAR.

FIG. 124.—STRUCTURE IN GREENWICH STREET.

derrick-car, and that the work could proceed nearly twice as fast with the latter.

The stations at 149th Street, 177th Street, and Fordham Road were reconstructed as express stations. Prior to the reconstruction work each had a center platform with two adjacent tracks, and, in addition, the station at 149th Street had a side platform on the east side of the east track, used for unloading the trains. The new express stations were constructed with two island platforms, with the center track between the platforms. In order to maintain the traffic, the work was carried out in the following manner: First, the new extensions to the cross-girders and the new outside track stringers were erected and the new tracks laid on top of these stringers. At 149th Street, where there was a platform on the east side of the structure, all this work had to be done under the platform without interfering with its use. The next thing to do was to extend the center platform over the existing running tracks, so as to be able to transfer the traffic from those tracks to the new ones. This was accomplished by building temporary platforms of the proper width on top of the new outside tracks, which, when completed, could be shifted over on top of the old tracks adjacent to the existing center platform, thus forming a new center platform of more than twice the width of the old one. The platforms were shifted, one at a time, during the early morning hours, when the traffic was so light that trains could run on a single track past the station, and was generally completed in about 1 hour.

All other stations between 149th Street and Fordham Road were local stations, but, as they had center platforms (with the exception of the station at 180th Street) which were in the way of the new center track, they were all reconstructed so as to provide side platforms to take the place of the existing center platforms. The cross-girders were extended, the extensions being erected with gin-poles, although the new platform girders were generally erected with the derrick-car.

Steel Erection.—Section No. 7.—The work on this section consisted of the reconstruction of the stations of the Ninth Avenue Line at 66th, 116th, 125th, and 145th Streets for express service.

The first station to be reconstructed was that at 116th Street. The additional column bracing was erected by placing light scaffolding around the columns and reinforcing the cross-trusses from hanging scaffolds. When this was completed, the erection of the new outside

track stringers was commenced. As the structure at this point is very high, and as a large number of stringers had to be erected, the usual method of erecting with gin-poles could not be used satisfactorily, and a derrick-car was built especially for the purpose. This car proved so useful that it was afterward used to erect steelwork at other points where conditions were favorable.

The new stringers were trucked to points in the street north of the station where the handling did not interfere with traffic in the street. The derrick-car, standing on the center track, which was not used for traffic, hoisted the girders, one at a time, through the space between the center and the south-bound tracks, placing them on the flatcar in front of it. When a girder was loaded on the flatcar, the construction train ran in on the running track immediately behind a passenger train, to the point where the girder was to be placed. The derrick-car was then blocked and guyed, the girder was placed, and the derrick-car continued south and took the cross-over to the center track, leaving the running track clear. The interval between the running trains was 3 min., and the work of placing the track stringers was completed so rapidly, that generally no delay was caused to passenger traffic. After having arrived on the center track south of the station, the train returned on the north-bound track to the center track north of the station, where another girder was picked up, and this continued until all the girders on the west side were erected. The girders on the east side of the structure were erected in a similar manner, except that the work train was loaded at the point and then ran directly over the south-bound track to the center track south of the station, and there awaited a favorable time to run in on the north-bound track to place the track stringers.

The new north-bound track in front of the north platform was then laid and connected to the existing local track, and the existing north platform was closed to traffic temporarily until the reconstruction of the platform was completed. The new north-bound track was supported on the new line of track stringers and the former outside track stringers of the abandoned local track. The inside track stringers, which no longer carried any load, were disconnected from the outside stringers, and those in front of the old platform were used for track stringers on the west side of the structure; the remaining ones were moved and used as platform stringers for the extension of the platforms.

In the meantime, the platform had been repaired and braced, and was then shifted bodily eastward to its new position. The shifting of the platform was done with jacks and steamboat ratchets. The track stringers removed from the east side were then placed to carry the east rail of the new center track, the west rail being carried by the existing east stringer of the old south-bound track. The center track past the platform was then laid, and temporary connections were made at both ends so that this track could be used for south-bound local service. Traffic was then transferred from the south platform to the new north platform, and the construction operation repeated. When this work had been completed, all the permanent track connections were made and both platforms were used, the north platform for north-bound traffic and the south platform for the south-bound traffic. The new mezzanine station was constructed in conjunction with the work on the track level.

The work on the station at 125th Street, in all general details, was similar to that at the 116th Street Station, the main difference being that the existing station buildings at the ends of the platforms were retained, instead of constructing a new mezzanine station under the structure, and that the cross-girders had to be provided with short extensions at each end to make them long enough to support the new lines of outside track stringers.

The stations at 66th and 145th Streets were of the hump type, and were erected in a manner similar to other hump stations. At 66th Street all work that could be carried out without interference with the center track was first completed, and the reconstruction of the center track was delayed, in order to have it carried out simultaneously with that of the center track at other points south of this station, so as to make the interference with the existing express service as small as possible. When the reconstruction was commenced, it was carried out and completed with an interruption of the express train service of only 2 weeks. The derrick-car was used to erect the towers supporting the new elevated center track, and a traveler erected the remainder of the structure. Two working shifts were used for a short time. The men commenced work at 3.30 A. M. and stopped at 8.20 P. M., in order to insure its completion within the prescribed time.

Steel Erection.—Section No. 8-A.—Section No. 8-A extended in Greenwich Street from south of Cortlandt Street to 9th Street, and the

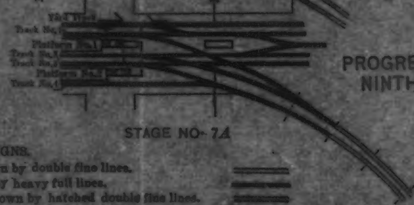
erection work consisted mainly of adding new cross-girders and center-track stringers to the existing structure. Fig. 124 shows a portion of the existing structure immediately after the erection of the new cross-girders and center-track stringers. It shows the shoring of the outside track stringers, which consisted of four-post framed towers built around the existing columns, where the columns were not removed, and of two similar towers, one at each side of the existing column, where the columns were replaced with new ones. The columns were generally removed at the new express stations, where the additional load of the express platform required heavier ones, and also where the existing columns were of the type shown to the left on Fig. 124. This, which was the original type of columns used on the elevated lines, consisted of four 6-in. I-beams tied together at intervals. The base of the column was an iron casting, into which the I-beams of the columns fitted and were held by rust caulking. Fig. 124 shows the base casting and a part of the column shaft, which was burnt in two to facilitate removal; it also shows the original track stringers on the west side of the structure, which were only about 24 in. deep, and, in the course of time, had been reinforced with another set of 24-in. stringers, placed directly on top of the original girders. These track stringers were removed after the center track was completed, the south-bound traffic being diverted temporarily to the center track.

At the stations which were converted for express service, the north-bound platforms and track were shifted a few feet east to make room for the additional track and platform. This was accomplished by supporting the ends of the track stringers to be moved on six-post shoring towers, wide enough to carry the stringers both in the original and the new position. As most of the cross-bents were on a skew, it was generally necessary either to cut off a portion of the track stringers moved or to lengthen them. In all cases the ends of the stringers were remodeled to connect to the new cross-girders or to the existing cross-girders in their new position.

The erection of the new cross-girder and track stringers was done with two travelers, one starting at the north end and the other at the south end of the section.

Steel Erection.—Sections Nos. 8-B and 8-C.—Section No. 8-B comprised the construction of the hump stations at 14th and 34th Streets on the Ninth Avenue Line, and Section No. 8-C comprised the elimina-

PLATE XVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1418.
GARDINER AND JOHANNESSON ON
ELEVATED RAILWAY IMPROVEMENTS.



STAGE NO. 7A

PROGRESS OF ERECTION
NINTH AVENUE LINE

STAGE NO. 1

1.—Track Layout same as at present.
2.—Place Cross-Girder Extensions 1236 E to 1252 E and 1236 W to 1252 W, removing Sidewalk Col. 1242 E, and place Outside Track Girders 1236 E to 1252 E and 1236 W to 1252 W, and Switch Tower at 1249 E.
3.—Place temporary Cross-overs as indicated.
4.—Lay Portions of Tracks Nos. 1 and 4 as indicated.
5.—Remove Passage Way to Polo Grounds and adjacent Yard Buildings.
6.—Erect temporary Signal Tower at Bent [illegible].

STAGE NO. 2

1.—Connect with old Track Portions of Tracks Nos. 1 and 4 new laid and use for Traffic.
2.—Shift old Track Girders on Center Line of Structure, Bents 1247 to 1250, closing Track No. 2 during this Operation.
3.—Lay Portion of Track No. 2 and connect to present Tracks as shown, and use for Traffic.
4.—Construct temporary Platform and Bridge from Bent 1254 to Bent 1252.
5.—Erect new West Sidewalk Columns and Cross-Girder Extensions from Bent 1258 to North End of Work.
6.—Raise present Track Girder at Bent 1276 and lay Track No. 1 north of Bent 1258.

STAGE NO. 3

1.—Use Temporary Platform for South-bound Traffic.
2.—Remove North foot Bridge and Remodel West Platform at North End.
3.—Raise present Track Girders and erect such new ones as will enable Track No. 1 to be completed from Bent [illegible] to 1258, and lay this Portion of Track No. 1.
4.—Place new Yard Track Girders at Bent 1258 and from Bents 1267 to 1276.

STAGE NO. 4

1.—Shift Girders [illegible] to 1267 from present West Track to new Yard Track and lay this Track.
2.—Construct Stations No. 1 and Parts of North and South Mezzanine Stations shown, and Stairways to Street.
3.—Raise Track No. 2 to new Position from Bents 1253 to 1258.
4.—Place Temporary Cross-over, as indicated.

STAGE NO. 5

1.—Stop Trains at temporary Station only.
2.—Raise Girders and construct Track No. 4 from Bent 1258 to North End and Track No. 3 from Bents 1252 to 1258.
3.—Place Girders that support upper Station, and construct part of Station shown.
4.—Place Structure on West Side of Putnam tracks north of Bent 1270 and lay tracks as shown ready for Connections.
5.—Lay Portions of Tracks Nos. 1 and 4, as indicated, ready for Connection.

STAGE NO. 6

1.—Remove temporary Cross-over between Tracks Nos. 1 and 2 in Spans [illegible] and [illegible] and finish the permanent work south of same and use Platform No. 1 for all Traffic.
2.—Finish [illegible] of Upper Station.
3.—Complete Tracks Nos. 1, 4 and 2 south of temporary Platform, eliminating Temporary Cross-overs and Switch Tower.
4.—Remove temporary Platform and East Platform south of Bent 1254 and finish Track No. 3 south of Bent 1250 and Track No. 4 between Bents 1251 and 1254.
5. Reconstruct West Station.

STAGE NO. 7

1.—Reconstruct East Station.
2.—Reinforce Last Cross-Girder Extension Bent [illegible].
3.—Complete Tracks Nos. 3 and 4 as far North as indicated.
4.—Construct Platform No. 2 and finish Mezzanine Stations.
Stage 7 A. Complete Tracks as indicated.
(a) Not to be started before N. Y. C. Putnam Division's Tracks and Platforms are abandoned.

CONVENTIONAL SIGNS.

Original Tracks shown by double fine lines.
Final Tracks shown by heavy full lines.
Temporary Tracks shown by hatched double fine lines.
Original Station Platforms, Buildings and Bridges shown by fine line outlines.
Final Station Platforms, Buildings and Bridges shown by heavy line outlines.
Temporary Station Platforms, Buildings and Bridges shown by hatched outlines.

tion of the express track grade crossing at 53d Street, where the local tracks of the Sixth Avenue Line connect with those of the Ninth Avenue Line. The erection of the steel for the hump stations was carried out in a similar manner to that described, and needs no further comment.

At 53d Street the approaches were erected in a similar manner to the hump stations, but the approaches were longer, because the head-room required under the new structure at 53d Street, where the south-bound Sixth Avenue track crosses under the new structure for the express track, necessitated a greater height of the express track over the local tracks than that required at the hump stations. On account of the crossings at 53d Street, the span length there had to be made about 99 ft., and the over-head track was carried on a through-bridge construction. The erection of the new structure was done with a derrick which, however, was not sufficiently strong to erect the through-bridge girders. Another and heavier derrick, therefore, was erected in the street at the west side of the structure, and was supported on towers in order to bring it to the level of the structure. Fig. 126 shows one of the girders in course of erection. The traffic on the lines was interrupted during the erection for a period of about 18 min. for each girder. After this span was erected, the large derrick was used to lift the traveler which erected the remainder of the work across the through span, which was too narrow for the traveler to pass through.

As stated when describing the erection of the 66th Street Station, Section No. 7, this station and the work under Sections Nos. 8-B and 8-C were carried out and completed at the same time, so as not to interfere with the express traffic more than necessary. This work, which involved the erection of 3 000 tons of steel and the removal or relaying of 7 000 ft. of track, was accomplished in 2 weeks.

Steel Erection.—Section No. 10-B.—Section No. 10-B comprised the reconstruction of the station at 155th Street and Eighth Avenue to serve the new express track, and the connection to the new rapid transit line in Jerome Avenue.

The erection problem was essentially one of maintaining traffic during the work. The method of doing this was worked out in advance, as shown on Plate XVIII, and was followed throughout, as far as it was completed, by the contractors. The final stages of the work were not completed by the contractors, as the east platform could not be built

before the Putnam Division of the New York Central Railroad vacated the space the platform was to occupy, and this again was dependent on other work on the connection, which was in progress.

Plate XVIII shows the different stages of the work. Stage No. 1 indicates the existing layout of the platform and track previous to reconstruction, to which were added a number of temporary cross-overs to divert the traffic from the tracks under reconstruction. During this stage, the cross-girder extensions and as many of the new outside track stringers south of the station as could be placed without interfering with traffic on the running tracks were erected, and a new temporary signal tower was built at 152d Street to take the place of the existing tower at the south end of the then existing west platform. This new signal tower was supported on timber bents from the street level. The track was laid as far as possible on the new track stringers south of the station, and, at the same time, the existing passageway from the west platform to the entrance to the Polo Grounds was removed. In the second stage, the newly laid portions of the tracks south of the station (marked No. 1 and No. 4) were connected to the old tracks, and the corresponding portions of the old tracks were abandoned. Such track connections were made without interruption of traffic, as they were generally completed in about 6 min. The existing center track, including the track stringers between 153d and 154th Streets, was then shifted west to its new permanent position (marked Track No. 2), and a new temporary platform was built between Tracks Nos. 2 and 4, connecting to the old east platform with a bridge over the running track. Fig. 127 shows the new platform and bridge in course of construction. At the same time the new columns and cross-girder extensions on the west side of the structure north of the old station were erected, and the new track (Track No. 1) on this portion was laid.

During the next stage (Stage No. 3) the new temporary platform was used in place of the old west platform, which was abandoned for passenger service, as well as the portion of the old west track in front of the platform, and this permitted the completion of Track No. 1 past this platform. During this stage, also, the new track stringers in the northerly stretch, for the new west track (marked Yard Track), were placed.

The remainder of the track stringers for this track were placed during the fourth stage, because they were old stringers, formerly

FIG. 125.—STATION ON THIRD AVENUE AT 9TH STREET DURING RECONSTRUCTION.

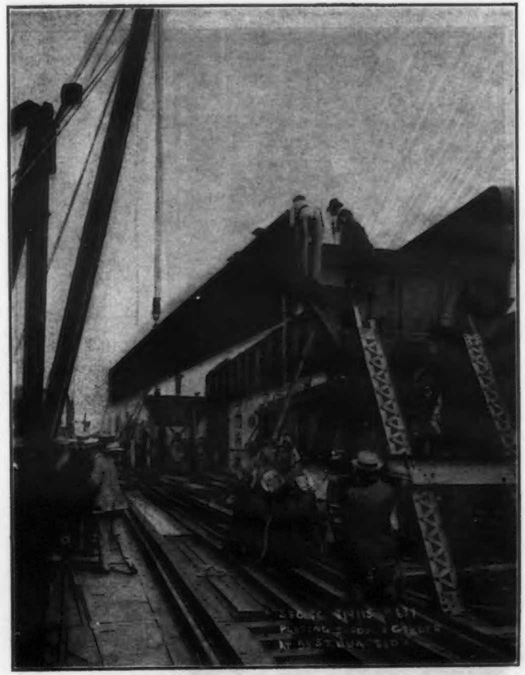


FIG. 126.—CROSSING ON NINTH AVENUE AT 53D STREET.

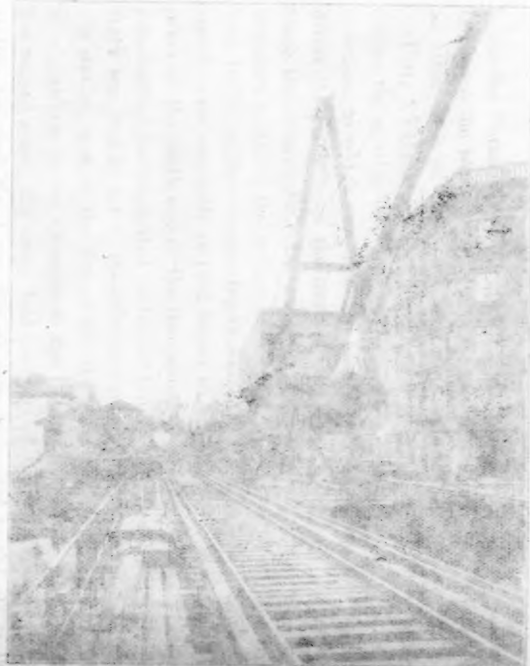


FIG. 127.—STATION ON EIGHTH AVENUE AT 155TH STREET. TEMPORARY PLATFORM.

FIG. 128.—STATION ON EIGHTH AVENUE AT 155TH STREET BEFORE RECONSTRUCTION.

used under the old west track, which, at this stage, was abandoned and replaced for traffic by the new Track No. 1. The new island platform (Platform No. 1) was then built, and also as much as possible of the new mezzanine stations at the north and south ends of this platform.

At the same time, Track No. 2, in front of the old station, was raised to its new level and connected with a temporary cross-over to Track No. 1, south of the new island platform.

During the next or fifth stage, the temporary platform south of 155th Street only was used for passenger traffic, and the two new tracks on the west side of the street (Track No. 1 and Yard Track), north of 155th Street, were used for connection to the yard at 159th Street. Track No. 2, north of 155th Street, was built to proper grade, and part of the upper station at 155th Street was constructed.

In the sixth stage, the new Platform No. 1 and the mezzanine stations were opened for traffic, and the temporary platform was abandoned, making room for the completion of the work south of 155th Street. The last stage can be completed without interfering with traffic, when the east side of the structure is abandoned by the Putnam Division.

Fig. 128 shows the original layout north of 155th Street. On the left side is shown the north end of the west platform and the passageway to the Polo Grounds; on the east side is shown the Putnam Division tracks and platform. Fig. 129 shows the same view after the passageway had been removed and a platform of the new steel on the west side of the structure had been erected. It shows the new track girders raised above the level of the existing tracks, which was done in order to obtain the necessary head-room for the mezzanine stations. Fig. 130 shows the new platform and upper mezzanine station completed for operation.

The complete reconstruction, as outlined, had to be carried out during the period between baseball seasons, as all the Major League games in New York City were played on the Polo Grounds, and by far the greater portion of the spectators at these games are handled through this station. The baseball season closed on October 20th, 1914, and the next season opened on April 13th, 1915. The reconstruction work started immediately after the close of the season, and

was completed by April 1st, 1915, but only by intense application throughout the whole period.

After this work was completed, the new over-grade express track between 150th and 155th Streets was erected with a traveler, starting at the south end of the incline.

STATIONS.

When passengers arrive at a station to take a train, they generally come singly or in small parties, and no delay is caused by passing through the station building to the platform. When, on the other hand, passengers are discharged from a train at a station, they are all ready to leave the platform at the same time, and, therefore, the exits to the street should be made as direct as possible. The stations remodeled or rebuilt under the "Manhattan Elevated Improvements" were designed with this principle in view. The exits were generally arranged to be as direct as possible, without any doors to obstruct the traffic.

Fig. 131 shows a diagram of the mezzanine station at Canal Street and the Bowery on the Third Avenue Line. From the four stairs connecting the mezzanine station to the street are passages leading to the ticket office and from there past the ticket boxes to the stairs leading to the platforms. The stairs which connect the mezzanine station with the platforms are used both for access to and exit from the platforms, as it is practically impossible to prevent passengers from using the stairs most convenient to them. Passengers leaving the platforms descend the stairs to the mezzanine station, and leave through the exit openings without interfering with the passengers coming in past the ticket booth. The mezzanine station is provided with toilet rooms, and only these and the ticket office are heated, as it would otherwise be necessary to provide doors, which would interfere with the progress of the passengers.

Fig. 132 is a plan of the mezzanine station at 125th Street on the Second Avenue Line. The general plan of this station is the same as that described, but, as traffic at this point is not so congested, the waiting-room is enclosed and heated. The exit is arranged direct, as at the Canal Street Station. It will be noted in both cases that all the toilet fixtures are backed against the walls of the heater rooms. This is done partly to keep the fixtures from freezing, but mainly to

FIG. 129.—STATION ON EIGHTH AVENUE AT 155TH STREET DURING RECONSTRUCTION.

FIG. 130.—STATION ON EIGHTH AVENUE AT 155TH STREET AFTER RECONSTRUCTION.

keep the pipes away from the toilet-rooms, so that they can be inspected and repaired without working in the toilet-rooms and also to prevent them from being destroyed by vandals. In fact, all the toilet fixtures are arranged so that nothing can by any ordinary means be broken or unscrewed and taken away; the flush-tanks, for example, are

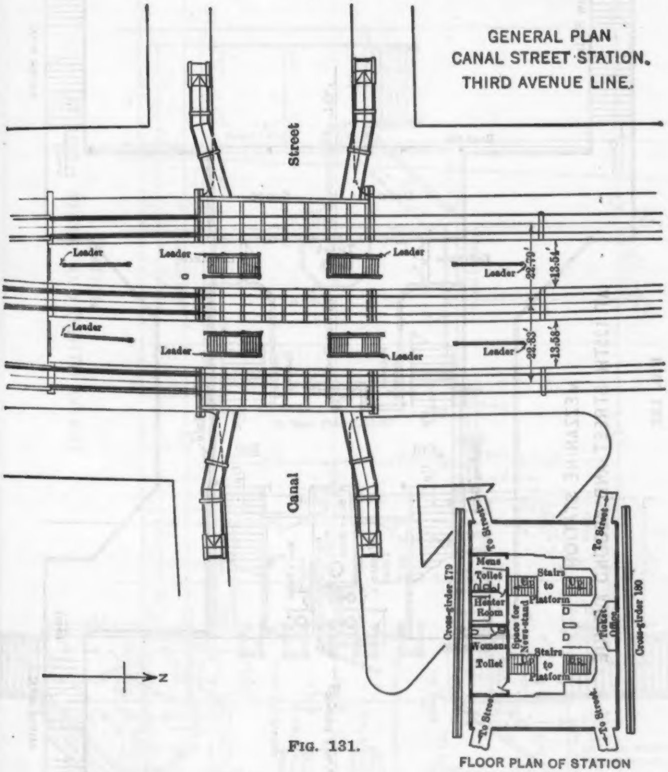
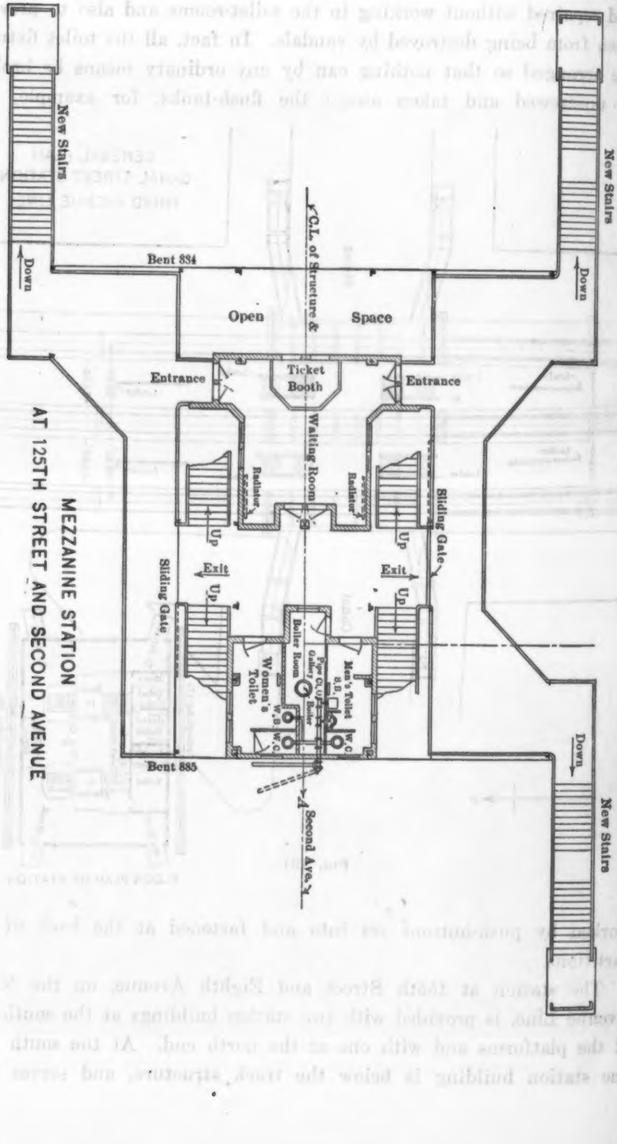


FIG. 131.

worked by push-buttons set into and fastened at the back of the partitions.

The station at 155th Street and Eighth Avenue, on the Ninth Avenue Line, is provided with two station buildings at the south end of the platforms and with one at the north end. At the south end, one station building is below the track structure, and serves pas-



MEZZANINE STATION
AT 125TH STREET AND SECOND AVENUE

FIG. 132.

sengers from Eighth Avenue; the other station building is above the track structure, and is connected with the viaduct forming the approach to the McComb's Dam Bridge across the Harlem River. The station at the north end of the platform is a mezzanine, and is mainly provided to give direct access to the baseball park at the Polo Grounds. Fig. 188 shows the arrangement of this mezzanine station, which was built to handle a large number of incoming passengers at the same time. The main stand of the baseball park [illegible] leading to the grandstands is at the same level as the mezzanine station and connected directly by a wide passage, on which a large number of ticket sellers and ticket choppers [illegible]. The stairs leading from the mezzanine station to the platforms are as wide as the width of the platforms and the whole layout would permit clearing the last [illegible] when only the west platform was [illegible] the same was handled without serious congestion, and when both platforms are [illegible] will be doubled.

Plate XIX shows a typical arrangement of a [illegible] station. The existing station buildings contained all the necessary facilities, and the addition was made as follows. [illegible] the station would, after buying and depositing his ticket, pass out on the existing [illegible] platform. If he desired to take a train on the upper level, he would ascend the stairs leading to the upper platform. These stairs are placed in such a way that they are [illegible] and there [illegible] access to the street.

The platform construction consists of [illegible] girders on top of which are placed I-beams, carrying the wooden floor-joists. Generally [illegible] and supported on a side [illegible] the latter. The [illegible] platforms [illegible] the length of the platforms [illegible] for the platforms, not all [illegible] joints. [illegible] laid out, the [illegible] spaced about 4 in. The canopies [illegible] for either part of their length or their full length, according to the traffic conditions and the location of the station buildings. The canopies were supported by posts. On side platforms usually two rows of posts were used, one being at the back of the platform and

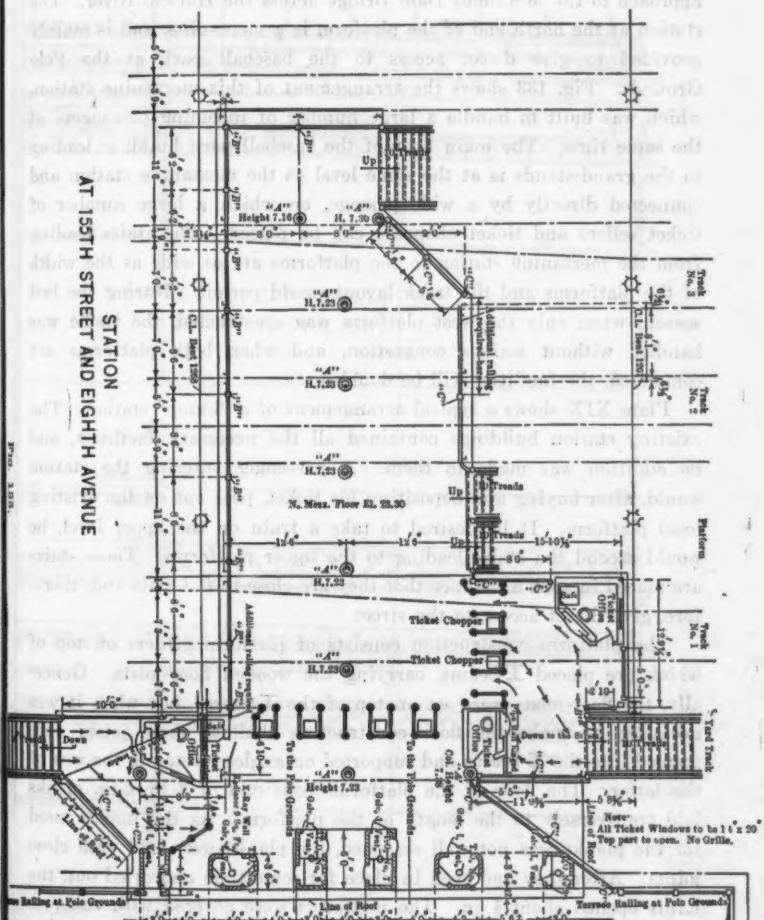


FIG. 188. STATION AT 155TH STREET AND EIGHTH AVENUE

sengers from Eighth Avenue; the other station building is above the track structure, and is connected with the viaduct forming the approach to the McCombs Dam Bridge across the Harlem River. The station at the north end of the platform is a mezzanine, and is mainly provided to give direct access to the baseball park at the Polo Grounds. Fig. 133 shows the arrangement of this mezzanine station, which was built to handle a large number of incoming passengers at the same time. The main floor of the baseball park building leading to the grand-stands is at the same level as the mezzanine station and connected directly by a wide passage, on which a large number of ticket sellers and ticket choppers can be placed. The stairs leading from the mezzanine station to the platforms are as wide as the width of the platforms and the track layout would permit. During the last season, when only the west platform was constructed, the traffic was handled without serious congestion, and when both platforms are completed, the facilities will be doubled.

Plate XIX shows a typical arrangement of a "hump" station. The existing station buildings contained all the necessary facilities, and no addition was made to them. A passenger entering the station would, after buying and depositing his ticket, pass out on the existing local platform. If he desired to take a train on the upper level, he would ascend the stairs leading to the upper platforms. These stairs are placed in such a manner that they are close to the exits and, therefore, give direct access to the street.

The platform construction consists of platform girders on top of which are placed I-beams, carrying the wooden floor-joists. Generally, the floor-joists were set on top of the I-beams, but when it was necessary to make the floor construction shallow, these joists were framed into the I-beams and supported on angles riveted to the web of the latter. The floor of the platforms consisted of 2 by 6-in. planks laid transversely to the length of the platform. As the lumber used for the planks was not well seasoned, the planks were laid with close joints. After they had been in place for some time and dried out, the joints opened about $\frac{1}{8}$ in. The platforms were covered with canopies for either part of their length or their full length, according to the traffic conditions and the location of the station buildings. The canopies were supported by posts. On side platforms usually two rows of posts were used, one being at the back of the platform and

PLATE XIX.
TRANS. AM. SOC. CIV. ENGRS.
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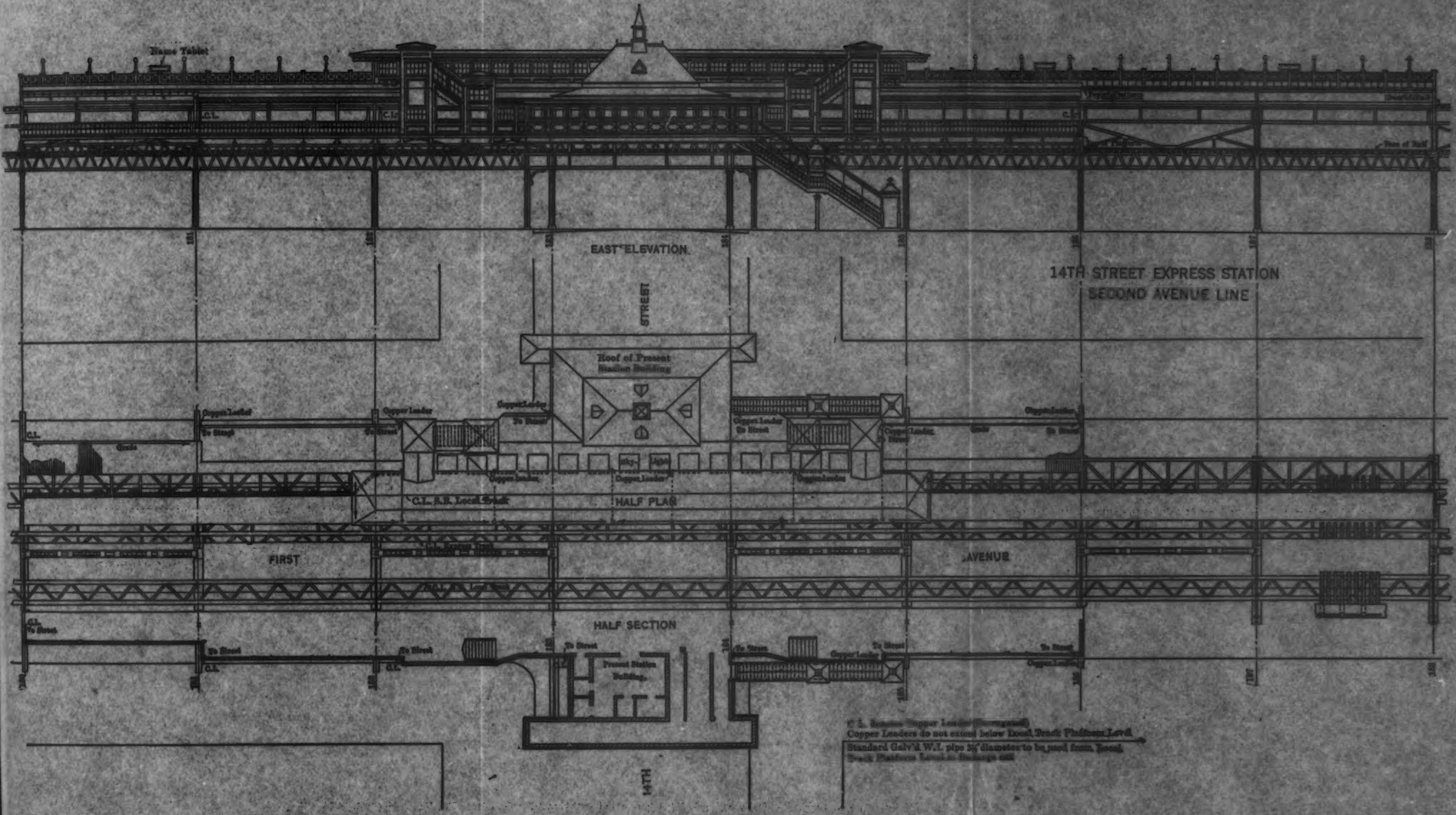
ELEVATED RAILWAY IMPROVEMENTS.



incorporated in the railing. The other row of columns was placed within a few feet of the front edge of the platform, far enough from the edge to make it safe to walk between the post and a train pulling in at the platform. On the center platforms, the canopy was usually supported by a single row of columns, set in the center so as to leave the platform as clear of obstructions as possible. The canopy was covered with tinned iron, laid on tongued and grooved boards.

The portion of the side platforms covered by canopies was usually protected with wind shields at the back of the platform. The wind shields were provided with detachable sash, so that the sash could be removed in the summer or when being cleaned.

The new station buildings were generally covered on the outside with sheet iron and painted, but, in some cases, on the down-town stations, copper was used. The inside woodwork trim was usually comb-grained yellow pine, producing a very pleasing effect.

Track Work.

Throughout the reconstruction work the standard Manhattan Railway track construction (Fig. 134) was used. The specifications for track timber were as follows:

"1.—The timber to be of long-leaved, first-growth, Florida, Georgia, or Alabama yellow pine, straight, square-edged, free from shakes, loose, large, or rotten knots, and every other material imperfection, planed on all sides, and of the full schedule dimensions after planing.

"2.—In no case will any stick be accepted with less than three heart corners, or with more than 1 in. of sap on the fourth corner, or more than 2½ in. of sap on either side at either end of the stick.

"3.—The timber to be delivered, as required by this Company, under the foregoing requirements, irrespective of trade usage or conventional specifications, and to be subject to the inspection and acceptance or rejection of the Company's authorized inspector."

The standard ties are 6 by 8 in. in cross-section, full dimensions, and are 8 ft. long. For the outside tracks, where a footwalk is required, every third tie is 12 ft. long and carries the footwalk. The standard spacing of the ties is 18 in. from center to center. The ties are fastened to the flanges of the track stringers with hook-bolts. The outside footwalk, laid on the 12-ft. ties, consists of five pieces of slatting, 2 by 6 in., full dimensions. Where there is a center track, the spaces between this track and the outside tracks are bridged over

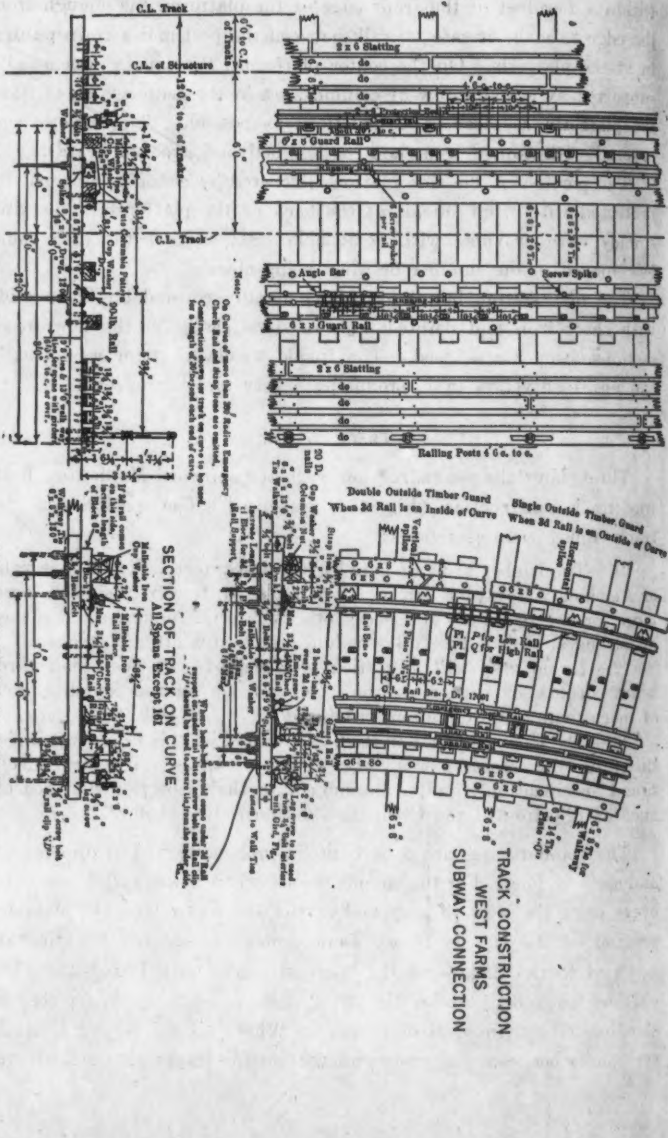

TRACK CONSTRUCTION
WEST FARMS
SUBWAY CONNECTION
SECTION OF TRACK ON TANGEN
SECTION OF TRACK ON CURVE
All Spans Except 161
SECTION OF TRACK ON CURVE
Span 161
C.L. Track
C.L. of Structure
2 x 6 Slatting
6 x 8 Guard Rail
Angle Bar
Screw Spike
Railing Posts 4' 6" c. to c.
Double Outside Timber Guard
When 3d Rail is on Inside of Curve
Single Outside Timber Guard
When 3d Rail is on Outside of Curve
90 lb. Rail

FIG. 134.

with standard ties, 4 ft. 6 in. from center to center, and provided with footwalks consisting of four pieces of slatting. The outside tracks are provided with four guard-rails of 6 by 8-in. timber, as shown, to prevent a train from falling off the structure in case of derailment. The standard track rails weigh 90 lb. per yd., have a 5-in. base, and are 5 in. high. On curved tracks the outer rail is super-elevated by elevation blocks, as shown by Fig. 134, and steel guard and check rails are provided. The contact rail, which weighs 100 lb. per yd., is supported on the footwalk ties. The outside footwalks are protected by pipe railings with posts 4 ft. 6 in. from center to center.

The track-laying could generally be carried out without interference with the traffic. Where track was replaced under trains in operation, the work was carried out by the Company's Maintenance of Way Department, which was accustomed to work of this kind.

Plant.

Yards.—The Company's material yard at 128th Street and the Harlem River, which, with the additional land and dock front leased for the use of the "Manhattan Elevated Improvements," had an area of about 50 000 sq. ft., and a dock frontage on the Harlem River of about 700 ft., was the main receiving and distribution depot for material used on the work. Most of the new steelwork for the up-town sections and the greater part of all rails, ties, and other track material, and also most of the scrap material from the structures removed, was handled through this yard. The equipment consisted of a derrick with a capacity of 10 tons, which was used for unloading steel from the lighters and loading it on the trucks, and a traveling derrick, running on rails on the surface of the yard, which was used mostly to handle the track timber. An existing derrick set on the elevated structure adjacent to the yard was used for handling the track material, which was distributed mostly with work trains. The steel was usually trucked to the working site and there picked up by the traveler erecting the structure. The yard was further furnished with a saw-mill and with the necessary storerooms and offices for the contractors and engineers.

Another, but smaller, material yard, on the Company's property at 133d Street, adjacent to the Third Avenue Line structure, was furnished with a blacksmith shop, and another small yard down town was leased for the storage of material, in order to avoid congestion in the streets.

A receiving and distribution point for the steelwork to be erected on the down-town sections was obtained by leasing a dock front and storage space at Perry Street and the North River, and, finally, a portion of the Company's surface yard at 179th Street and Third Avenue was used as a storage and distribution yard for material used on Sections Nos. 6-C and 7.

Compressors.—All riveting, drilling, etc., was done by compressed air, three stationary and thirteen portable compressors being used. The stationary compressors were Ingersoll-Rand, driven by a Nagle locomotive-type, 100-h.p. boiler; the portable compressors were Chicago Pneumatic Tool Company or Ingersoll-Rand compressors, with a capacity of 300 cu. ft. of free air per min. at a pressure of 90 lb., driven by 50-h.p. electric motors, taking current from the contact rail of the existing railways.

Travelers.—Although in some cases the erection of the steel structure was done with gin-poles and jinnywink, the greater part of the erection was done with travelers. Figs. 109 and 125 show the two general types used. When the width of the structure permitted it, the type shown by Fig. 109 was used. It spanned over two tracks, and was either supported on skids or on wheels running on a temporary track. When the working space was confined to the limits of the center track, the traveler was made narrow enough to be supported on the track stringers of the center track only, as shown by Fig. 125. The safe swing of the boom was limited in this case to the space between the outside running track, and necessitated picking up the steel between the running tracks. In order to prevent lateral swinging of the boom beyond the safe limits, which might endanger the stability of the traveler as well as the trains on the running tracks, wire ropes were attached half way up to the boom and connected to the frame of the traveler, which prevented the boom from swinging beyond certain limits. The hoists of the traveler were generally operated by electric motors, receiving the current from the contact rail of the railroad track.

Some of the steelwork was erected with a derrick-car built especially for the purpose. The derrick was built on one of the Company's standard flatcars. The A-frame was made so that it could be folded down on the derrick-car, in order to clear overhead obstructions while being transported from point to point. While being thus transported

the boom rested on another flatcar coupled in front of the derrick-car, and this flatcar was also used to transport the steel to the point of erection. The carrying capacity of the derrick-car was 8 tons, with the boom out to the side at an angle of 45 degrees.

CONTRACTORS AND COST.

Selection of Contractors.—When the work under the "Manhattan Elevated Improvements" was finally authorized to proceed, it was the desire of the Company that it should be carried out and completed speedily, safely, and without interference with the service of its trains. To secure these conditions, it was necessary to select contractors whose standing and experience would assure the success of the work, and, in order to retain their services, it was necessary to pay them for their work in an equitable manner. To advertise for bids, either on a lump-sum or unit-price basis, and then accept the lowest bidder would be a gamble, as far as the selection of the contractors was concerned, and the odds would be in favor of getting inexperienced contractors, as such men are not always able to foresee and provide against all the contingencies of the work. Even if the service of experienced contractors were secured, for instance, by accepting bids which were not the lowest, the Company would not have the freedom to select when, where, and how, the work should be carried out, so as to suit the traffic conditions on the railroads. The contractors would object to such interference, and properly so, as it would, at least in their opinion, decrease the efficiency of their work and the size of their expected profits. As an example of how the traffic conditions determined the conditions of the work, it may be mentioned that the station at 155th Street and Eighth Avenue serves the Polo Grounds, where the Baseball League games are played; the work of reconstructing this station could not commence until the baseball season was over, and had to be completed before the next season started. Another example is the carrying out of the work in Ninth Avenue; there used to be on the Ninth Avenue line a partial express service as far south as 14th Street; in order to interfere as little as possible with this service, the Company desired that the interruptions of the express service due to the reconstruction of the stations at 66th, 34th, and 14th Streets, and the grade-crossing elimination at 53d Street, should be simultaneous, and should not last more than 14 days. The work was carried out as, and within the time, described by

the Company, but not, from a contractor's point of view, in the most profitable manner. Again, when the three spans of the Harlem River Bridge were placed, it was ordered that the work should be done on a night between Saturday and Sunday after midnight, at which time the traffic was lightest; as the work depended on the tide, the placing had to wait until a Saturday night when the tide was favorable; in fact, throughout the construction, the traffic conditions were the determining factors in the progress of the work.

The desire of the Company to have the work completed speedily (the importance of which is shown by the fact that the number of passengers carried during the first year after the improvements were completed and in operation increased more than 13% over the number carried in the preceding years, during several of which the number had remained nearly constant) necessitated multiplications of the plant, tools, and materials for falsework, which contractors on a lump-sum or unit-price basis would be extremely unwilling to furnish, on account of the heavy additional cost.

Finally, the desire of the Company to carry its passengers safely during the work of construction, under contracts on a lump-sum or unit-price basis, would lead to endless controversies between the Company and the contractors, as to what constituted safety, both as to temporary supports and as to methods of conducting the work, while it was properly the intention of the Company to be the sole judge in the matter of safety.

The Company, therefore, elected to choose contractors who were known to it to possess the necessary experience to carry out the work, and to pay them the actual cost of the work plus a fixed percentage of the same to cover the use of their plant and their services. This method of payment was the only equitable one that could be devised for a construction work of this character.

The soundness of the Company's decision is shown by the facts that the work was completed in 23 months, without undue delays of the train service, and practically without injuring a passenger, although more than 600 000 000 passengers were transported on 2 000 000 trains over the lines under construction.

The work was performed under a contract, dated February 13th, 1914, with the Terry and Tench Company, Incorporated, The Snare and Triest Company, and the T. A. Gillespie Company, which last Company acted as executive. The work was distributed as follows:

All foundation work was done by the T. A. Gillespie Company; the Snare and Triest Company carried out all work, including steel erection, station finish, and track-laying on Sections Nos. 6-C and 7; the Terry and Tench Company completed all steel erection and track work on the remaining sections, and the station finish work on these sections was partly carried out by the Terry and Tench Company and partly by the T. A. Gillespie Company. The contractor's work was in executive charge of a vice-president of the T. A. Gillespie Company.

Structural Steel.—The sub-contractors for the manufacture and delivery of the steelwork, the tonnage delivered, and the prices per pound of the material delivered, are given in Table 1.

TABLE 1.—STEELWORK FOR "MANHATTAN ELEVATED IMPROVEMENTS."

Section.	Sub-contractor.	Tons.	Price per pound.
1	Milliken Brothers	5 963	0.0288
2-A	American Bridge Company	4 562	0.0293
2-B	American Bridge Company	3 820	0.0262
3	Phœnix Bridge Company	8 929	0.0239
4-A	McClintic-Marshall Company	5 544	0.0215
5-A	Pennsylvania Steel Company	1 732	0.0290
5-B	Pennsylvania Steel Company	1 440	0.0275
5-C (Structural steel)	Pennsylvania Steel Company	1 100	0.0290
5-C (Machinery)	Pennsylvania Steel Company	163	0.1045
5-D	Pennsylvania Steel Company	1 113	0.0320
6-A	L. F. Shoemaker and Company	2 926	0.0250
6-C	McClintic-Marshall Company	8 075	0.0243
7	L. F. Shoemaker and Company	1 487	0.0254
8-A	American Bridge Company	5 041	0.0245
8-B	Milliken Brothers	1 802	0.0270
8-C	Milliken Brothers	547	0.0254
10-B	Belmont Iron Works	1 200	0.0285

Rails.—The contracts for the manufacture and delivery of rails were made with the following companies:

Bethlehem Steel Company, standard rails, 2 961 520 lb., at $0.014 per lb.

Lackawanna Steel Company, standard rails, 2 916,940 lb., at $0.014 per lb.

Illinois Steel Company, manganese rails, 564 060 lb., at $0.0905 per lb.

Lumber.—The track lumber, which was all yellow pine and amounted to about 8 000 000 ft., b. m., was obtained from the D. L. Gillespie Company, of Pittsburgh, Pa., and cost $30.50 per 1 000 ft., b. m.

Other lumber, for shoring, forms, etc., was bought for $23.50 per 1 000 ft., b. m.

Material for Concrete.—The prices paid for materials for concrete delivered were as follows:

Cement	$1.70 per bbl.
Sand	1.00 per cu. yd.
Stone	1.70 " " "

Cost of Labor.—The wages paid to the men engaged on the work were at the prevailing rates, and were as follows, for an 8-hour day:

Bricklayers	$6.00
Iron workers	5.00
Iron workers' apprentices	3.00 to $4.00
Carpenters	5.00
Carpenter foreman	7.00 to 8.00
Dock builders	4.00
Water-proofers	4.25
Rock drillers	3.75
Timber men	2.50
Timber men or foreman	3.00 to 3.50
Painters (structural)	2.25 " 2.50
Painters' foreman	4.00 " 5.00
Blacksmiths	4.50 " 6.00
Blacksmiths' helpers	3.20
Machinists	3.50
Lead caulkers	4.50
Labor foreman	3.50 to 4.00
Handy man	2.50 " 3.00
Laborers	2.00 " 2.25
Painters for timber work	4.00
General foreman	8.00 to 10.00, straight time.
Compressor men	125.00 per month.
Hoisting engineers	30.25 to 33.00 per week.
Watchmen	12.00 " 14.00 per week.

Total Cost.—The total cost of the work done by the contractors was $10 273 636.98, distributed on the sections as follows:

Section.	Cost.
1	$1 031 678.76
2-A	773 900.88
2-B	656 784.27
3	717 651.03
4-A	692 453.22
5-A	360 804.42
5-B	325 669.73
5-C	258 458.71
5-D	187 341.30
6-A	602 671.68
6-C	1 488 380.41
7	590 966.43
8-A	854 004.07
8-B	356 153.48
8-C	108 153.48
10-B	456 919.91
General	811 196.85

The term "General" includes charges which could not be definitely distributed on the various sections. The contractors' expenses for engineering and superintendence are included in this item, and amount to $559 340.45, or about 5½% of the total cost.

Cost of Steel Erection.—The cost of steel erection varied greatly for the different sections, on account of the varying difficulties connected with the erection. Table 2 gives for each section the cost of erecting 1 ton of steel and the cost of shoring per ton of steel erected.

TABLE 2.—COST OF ERECTION.

Section No.	Cost of erection, per ton.	Cost of shoring, per ton.
1	$30.42	$12.57
2-A	21.27	5.41
2-B	13.80	1.50
3	43.71	20.18
4-A	14.22	0.88
5-A	50.85	20.61
5-B	39.62	26.53
5-C	85.76	....
5-D	32.49	6.49
6-A	28.70	17.06
6-C	39.82	10.30
7	93.94	7.34
8-A	83.41	15.33
8-B	....	....
8-C	57.80	13.90
10-B	105.24	2.68

The cost of raising the structure in Second Avenue and 125th Street, as described under Section No. 5-B, was \$6 812, and the cost of moving the tracks between 133d and 144th Streets, as described under Section No. 6-A, was \$8 474.

As an example of the cost of riveting, etc., it may be stated that, on Section No. 6-C, 365 000 rivets were driven at the cost of 14.2 cents per rivet, including overhead charges. There were four men in a gang, and each gang averaged 172 good rivets for an 8-hour day. This work was interfered with, on account of the train traffic. About 325 000 holes were drilled from the solid, mostly $\frac{15}{16}$ in. in diameter, at a cost of 15.1 cents per hole; about 40% of these holes were drilled in new steel on the street surface, the remainder in the structure. About 115 000 old rivets and bolts were cut out of the existing structure at a cost of 9.6 cents each, including replacing the rivets cut out with temporary bolts.

Cost of Placing Foundations.—The labor charges for the construction of the foundations varied considerably, according to whether the new foundations were at new locations or replaced existing foundations, and also according to the sub-surface structures encountered. On Section No. 2-A, for example, 49 foundations, each containing, as an average, 15.8 cu. yd. of concrete, were placed at new locations at \$24.50 per cu. yd., including all excavation, placing of sheathing, forms, and concrete, back-filling, and repaving. Seventeen foundations under existing columns were placed at a cost of \$37.00 per cu. yd., and, in addition, the cost of shoring the structure amounted to \$210.36 for each foundation.

On Section No. 5-A, 23 foundations, each containing, as an average, 20 cu. yd., under existing columns, were placed at a cost for labor of \$18.93 per cu. yd., and the charges for shoring the structure amounted to \$255.77 for each foundation.

On Section No. 5-B, 24 foundations, each containing 7.5 cu. yd. of concrete, at new locations, were completed at a cost of \$22.47 per cu. yd., or a total of \$168.50 per pier, distributed as follows:

Superintendence:	Timekeeping .... Storekeeping .... Material checking General foreman..	\$21.02
	Carried forward..................	\$21.02

	Brought forward..................	$21.02
Carpenter work:	Protections	4.00
	Sheathing	20.00
	Making forms..............	4.00
	Placing "	7.00
	Stripping "	3.40
	Repairing "	6.00
Labor:	Excavating	50.80
	Concreting	9.90
	Back-filling	10.16
	Cleaning up..............	5.00
Watching.		12.10
Hauling materials...........................		15.12
		$168.50

On Section No. 1, 50 foundations, averaging 242 cu. yd. of concrete, under present columns, were placed at a cost of $22.73 per cu. yd., and the shoring labor cost $133.43 per column.

The labor charges for moving the sub-surface structures, as described under Section No. 2-A, amounted to $21 760, and, as described under Section No. 2-B, to $99 931.

Cost of Track-Laying.—The labor cost of laying center track on Section No. 6-C amounted to $1.90 per lin. ft. of straight track and $2.40 per lin. ft. of curved track.

On Section No. 2-A, 9 333 ft. of track were laid at a total cost of $20 918.28, or $2.25 per lin. ft., and on Section No. 2-B, 8 496.5 ft. of track were laid at a total cost of $20 082.98, or $2.37 per lin. ft.; these last prices include the railing for the footwalks.

Accounting.—The method of accounting for the expenditures, complying with the regulations of the Public Service Commission, was as follows:

Pay-rolls rendered weekly by the contractors were checked by daily force account showing the men's names, numbers, and occupations. The time was taken by timekeepers of both the contractors and the Company, who signed the sheets each day. The daily average reports also appeared on the time sheets. The time of all employees was also checked in the field by the representative of the Public Service Commission.

Material delivered on the job was entered on daily receiving sheets, being charged directly or entered into stock and charged to the job when drawn out on warehouse orders, showing the job number to which it was to be charged. The same rule as to distribution applied to the direct charge reports; all reports were signed by the engineer in charge or a duly authorized assistant. Materials taken into stock were entered on a monthly stock report furnished by the contractor and checked by the engineer.

All accounts of the contractor for pay-rolls and material received were paid for through a voucher system, sent to the engineer for checking, and the monthly estimates from which the contractor received his compensation were made up from the vouchers.

Accidents.

In addition to the usual hazards connected with erecting a steel structure, the working conditions were subjected to the dangers of frequent and fast-moving trains and of live high-voltage contact rails. A careful record was kept of all accidents that occurred during the work. The timekeepers were instructed to report immediately, on special accident report forms, all accidents that occurred, even the most trivial ones. During the whole period from March, 1914, to February 1st, 1916, when the work of the contractors was completed, there was reported 3 334 accidents, out of which 26 were fatal. As there had been in that time 1 060 000 single man working days, the average amounted to 1 accident daily per 318 men. No analysis has as yet been made of the total number of accidents, but, in April, 1915, when 1 681 accident reports had been received, which is practically one-half of the total number of reports received, an analysis was made of the accidents reported up to that time. It was found that, as an average, an accident happened daily to 1 man out of 300 employed. Of these accidents 71% did not cause any loss of time to the employee hurt and 88% (which includes the 71% mentioned) returned to the work in 2 weeks or less from the time of accident. Ten accidents were fatal.

The accidents, in accordance with their general character and their frequency, may be classified as follows:

Bruises, sprains, etc.	40 per cent.
Cuts	34 "

Accidents to the eye	12	per cent.
Accidents caused by projecting nails	6	"
Fractures	4	"
Burns	4	"

Under bruises, sprains, etc., are included all accidents resulting from having a hand or foot squeezed by materials handled, etc. Under cuts are included all open wounds, particularly those inflicted by sharp tools. Accidents to the eye include mainly small particles of foreign matter getting into the eye. The accidents under the headings, "accidents caused by projecting nails" and "fractures" are self-explanatory; the accidents under the heading "burns" were chiefly caused by short circuits from the contact rail.

As far as the causes of the accidents are concerned, they may be classified as follows:

Self-inflicted accidents	56	per cent.
Accidents caused by fellow employees	33	"
Accidents caused by trains and street cars	2	"
Accidents caused by material dropping from the structure	4	"
Accidents caused by failure of plant	3	"
Accidents caused by short circuit	2	"

The accidents included under the heading "self-inflicted" comprise all such accidents as were caused by the injured persons themselves; and, under the heading "accidents caused by fellow employees" are included accidents which occurred while the injured persons were working in conjunction with other workmen. The other headings explain themselves, except the expression "failure of plant". The greater part of the accidents under this heading are such as were caused by the slipping of a ladder or the blowing out of the snap of a riveting hammer. Properly speaking, such accidents are as much failure of plant as the dropping of a boom or the breaking of a scaffold.

The following means were used to prevent accidents, as far as possible. Every employee, before his services were accepted, was examined carefully by physicians, retained by the Company for the purpose, to assure that he was physically fit for the work; the foremen

were instructed to discharge any employee showing carelessness and disregard for his own or others' safety; employees under the influence of liquor were discharged and not re-employed. Flagmen employed by the Company and not by the contractors were stationed at all points where employees were on or near the running tracks; their duty was to see that all workmen were clear of the track before a train was permitted to pass. Where the structure was of unusual height, canvas or wire nets were stretched under it at the points of work. All metal tools used in the vicinity of a live contact rail were wound with insulating tape to minimize the danger of short circuits.

By far the greater number of accidents happened to the hands and feet of the employees while handling material. It would seem that such accidents could be materially reduced if only the employees could be persuaded to use proper caution. There is another class of accidents, which includes a considerable percentage of the cases, the prevention of which may be more definitely charged to the superintendence. These accidents are such as are caused by loose articles dropping from the structure, by projecting nails, and by men getting particles into their eyes when drilling steel or doing similar work. Loose articles should be secured, or picked up and placed in proper receptacles; projecting nails should be hammered down and made harmless, and drillers and others exposed to dust should wear proper protections for the eyes. It may be stated that inspections as rigid as possible along these lines were made throughout the progress of this work.

It will be noted that during the first half of the work accidents occurred at the rate of one a day for each 300 men working, but, during the latter half, they occurred at the rate of one a day for each 340 men working. Unless this in itself is an accident, it may indicate that the experience gained during the work decreased the chances for accidents occurring, and increased the knowledge as to how to prevent them.

During the work 26 fatal accidents occurred to men engaged thereon, one of which happened to one of the Company's engineers, who was struck by a train while performing his duties. Some of the accidents happened while the employee was not immediately engaged on his work, and not a single one happened when the work might be considered extra hazardous; in fact, very few accidents, fatal or other-

wise, happened when those engaged on the work realized that special precautions had to be taken. Here it may be mentioned that the placing of the three spans of the Harlem River Bridge, which work was done during the night, and once during a heavy snow storm, was completed without a single, even minor, accident, although the work, on account of the interruption it caused to traffic, was performed at the highest possible speed.

In addition to the accidents to the employees, a number happened to people passing in streets in the vicinity of the work. The greater number of these were caused by people colliding in various manners with obstructions in the streets. Other accidents were caused by material falling from the structure. The actual number of accidents to people in the streets is difficult to ascertain, as the claim that an accident had happened did not always necessarily mean that it did. There were very few and minor accidents to passengers on the platforms or in the trains.

The safety of the people in the street was provided for, as far as possible, by guarding all work with fences, by placing warning signs on all obstructions, and by stationing flagmen at all points where their services might be of value, and whenever possible by placing canvas protections under the structure, to guard against material falling to the street. The train service was protected by flagmen provided by the Company; their duty was to see that the line was clear before a train was permitted to proceed.

Organization.

The whole work, including the complete design and all the field work, was done under the direction of George H. Pegram, President, Am. Soc. C. E., Chief Engineer of the Interborough Rapid Transit Company, and was carried out in direct charge of F. W. Gardiner, M. Am. Soc. C. E., Principal Assistant Engineer, with S. Johannesson, M. Am. Soc. C. E., as General Assistant Engineer. H. C. Soest, Assoc. M. Am. Soc. C. E., was in charge of the field work of Section No. 1; H. Leser, Assoc. M. Am. Soc. C. E., of Sections Nos. 2-A, 2-B, 3, and 4-A; Mr. B. O'Rourke of Sections Nos. 5-A, 5-B, 5-C, and 5-D; Mr. J. M. Jorgensen of Sections 6-A and 6-C; Mr. O. Whitman of Sections 7, 8-C, and 10-B; and Mr. B. Helseth of Section No. 8-A.

For the contractors, Edward J. Govern, M. Am. Soc. C. E., was in general charge of the work. Holton D. Robinson, M. Am. Soc. C. E., was the contractor's Engineer, in charge of the work done by the Terry and Tench Company, and W. P. Rothrock, M. Am. Soc. C. E., of the work done by the Snare and Triest Company.

It may be stated that, throughout the work, the Engineers of the Public Service Commission, as well as the Contractors, co-operated with the Engineers of the Company to bring it to a successful finish, and that the Transportation Department of the Company, under Superintendent S. D. Smith, and the Maintenance of Way Department, under C. E. Carpenter, Assoc. M. Am. Soc. C. E., gave valuable assistance in carrying out the work.

DISCUSSION

CLARENCE E. CARPENTER,* ASSOC. M. AM. SOC. C. E.—The authors of this paper are entitled to the thanks of the Profession for having prepared and presented a detailed and complete description of one of the most difficult and extensive traction developments ever carried out in New York City. The work was divided into sixteen sections, according to location on the several elevated lines. Each section presented, to a greater or less extent, distinctive problems calling for a special design and method of procedure. The paper, therefore, is of considerable value for reference to engineers who may be called on to execute work of this character. Mr. Carpenter.

The speaker's connection with the work was practically limited to the maintenance and operation of the elevated lines during its progress and after completion. Detailed plans and specifications for track construction were prepared by the Maintenance of Way Department of the railway company.

The greater part of the track material was loaded on work trains at the storage yards and distributed along the right of way by that Department. All track work, within the clearance lines of tracks under operation, was performed by the company's maintenance force, and also all contact-rail work and accessories. All flagmen stationed on the tracks to prevent trains from running into obstructions or striking employees were furnished from the same force. The flagmen did their work well, as on no occasion did a train encounter any obstruction or become derailed due to the track not being clear or in proper shape for operation. The Maintenance of Way Department furnished experienced ironworkers for the inspection of rivets, and inspectors who co-operated with the construction engineers in following up the contractor's excavating and shoring work to insure the stability of the structure and the safety of passengers. At times during the progress of the work as many as 150 men were employed as flagmen and rivet inspectors.

The work comprised the laying of approximately 24 miles of single track, of which 15 miles were new track added to the elevated system and 9 miles were old track rebuilt. About 6 500 tons of new steel rails and 12 000 000 ft., b. m. of new track ties were used.

It may be of interest to refer to a few of the important features of the track construction. All track rails on tangents, and on curves having a radius larger than 400 ft., were of open-hearth steel having the following chemical composition:

Carbon............0.75 to 0.90; average not less than 0.80;
Manganese........0.60 to 0.90;
Silicon, not more than 0.20;
Phosphorus, not more than 0.04.

* New York City.

Mr. Carpenter.

All rails on curves having a radius of 400 ft. and less, except in yards and emergency cross-overs, were of manganese steel. Steel guard-rails were placed on the inside rail of all curves of less than 2 000 ft. radius. The guard-rails were bolted to the track rails at intervals of 3 ft. with $\frac{15}{16}$-in. steel bolts, and were carried at least 60 ft. beyond the points of tangency of the curves. All rails on tangents through switches and special work, and on curves having a radius of 500 ft. or more, were laid true to gauge of 4 ft. $8\frac{1}{2}$ in.; on curves between 500 and 150 ft. the gauge was widened to 4 ft. $8\frac{3}{4}$ in.; and on curves having a radius of less than 150 ft. the gauge was 4 ft. 9 in.

The replacement of the old Second Avenue-Harlem River Drawbridge with a new double-deck four-track drawbridge was perhaps the most conspicuous feature of the work. The bridge has two shore spans and one draw-span. The authors have described the replacement of the south shore span on February 22d, 1915, and of the north shore span on March 6th, 1915. The excessive variation in the range of the tide in the Harlem River caused some anxiety during both these operations. In the case of the removal of the old south shore span, the tide reached its highest level and the northeast corner of the span still rested on the pier. A derrick boat was used to lift that corner, and, fortunately, the span just swung clear of the piers. At the time the north shore span was replaced, the range of tide was nearly twice as great as when the south span was floated. The scows were placed under the old span and the blocking set up to lift it at 1 A. M., when arrangements had been made to suspend traffic on the bridge. The tide rose with great rapidity and began to lift the span, so that trains had to be flagged at both ends of the bridge at 12.44 A. M. Before the draw-span could be opened, the shore span rose so high that the lift-rails would not swing clear, and some very lively work was necessary in order to disconnect the tracks before damage resulted.

Mr. Pegram.

GEORGE H. PEGRAM,* PAST-PRESIDENT, AM. SOC. C. E.—This paper is so complete that the speaker did not expect to discuss it, but Mr. Kittredge has alluded to the co-ordination of forces in work of this kind, and it is proper to state that the co-ordination practiced was in line with what the different governments of the world are now finding to be more and more necessary: the centralization of power in one man.

After the preparation of the general plans, Mr. Gardiner was given, practically, a free hand in the preparation of detailed plans and in carrying out the work of the structural portions, and C. E. Carpenter, Assoc. M. Am. Soc. C. E., Engineer of Maintenance of Way, was given similar freedom in the necessary track changes and extensions.

In viewing the work at the beginning it did not seem possible that it could be accomplished without accident. It certainly did not seem

* New York City.

probable that it could be accomplished without more interruptions of traffic than occurred, and this was largely due to a centralization of power, which prevented confusion of orders. Mr. Pegram.

The reference which Mr. Thomson has made to the ancient history of the structures is of interest. These old elevated railways are probably the oldest metal structures in the world in point of use, having been subjected, during the 40 years of their existence, to constant and rapidly repeated loads, in some cases exceeding those for which they were designed.

Great credit is due to the designers and builders who produced a structure which after 40 years could have a third track added without the entire replacement of the old work. There were many details in the old structure for which there was no precedent. The one requiring the greatest courage in design was the single-column structure. The designers probably did not anticipate that the swaying of the locomotives would cause a lateral thrust of 6 000 lb. at the top of each column; and they probably did not anticipate that the extensions of the station platforms would be supported on cantilevers from the columns, thus causing a bending moment.

Although some of these columns rest on sand, no undue or irregular settlement has occurred. The columns were inserted in sockets in a cast-iron base, in which they were fixed with a "rust" joint. This detail has proved very effective, and could well be repeated in the future. There are a number of columns with closed section in which no corrosion has been found after their 40 years of service, which is proof that the old and persistent specification, that closed sections should not be used, is unwarranted. A closed section is the strongest one for a column. There is no reason for not using it more generally.

Of course, the circulation of air in the columns of the elevated road is prevented, but it might also be prevented in the columns of bridges.

The detail most criticized is the want of intersection of the web members of the lattice girders. This is a very crude detail, and was probably adopted because the material in the chord was supposed to be so excessive that it was not necessary to take care of the moments at the joints; but this proved wrong, and it was necessary to double-lattice the girders to relieve these moments.

Cast iron is used for the supports of the canopies of the platforms and stairways of the old elevated structure. This is a detail which could be followed with advantage, as the cast-iron column can be made smaller, more ornamental, and less dangerous through contact than one made of rolled shapes.

T. Kennard Thomson,* M. Am. Soc. C. E.—This Society and the City of New York owe a very great debt of gratitude to the authors and to their Chief, George H. Pegram, Past-President, Am. Soc. C. E., Mr. Thomson.

* New York City.

Mr. Thomson. Chief Engineer of the Interborough Rapid Transit Company, both for this comprehensive paper and for the marvelous success with which they replaced the old elevated structures.

The old elevated railroad was being loaded far beyond the expectations of the original builders, and was rapidly becoming absolutely unsafe.

The new structures are thoroughly substantial and up-to-date, and show many ingenious methods of overcoming unusual difficulties of design; only engineers can realize the great care and engineering skill which Mr. Pegram, Mr. Gardiner, and their staff exercised so well, but withal so quietly and unostentatiously, in overcoming these unprecedented difficulties. This achievement should be brought to the attention of the public.

The speaker hopes that Mr. Pegram will enhance this valuable paper with many historical and other remarks about the old elevated railroad.

Fig. 135, showing a proposed elevated railroad for New York City in 1833, or some 30 years prior to the building of any structure of that kind, may be of interest.

Mr. Wegmann. EDWARD WEGMANN,* M. AM. SOC. C. E.—The speaker wishes to compliment the authors on the excellent paper they have given the Society. It is so full of details, and involves so much complicated work, that it can only be discussed properly by an expert bridge engineer who is thoroughly familiar with the local conditions.

As the authors have made a brief reference to the early history of the elevated railways of New York, the speaker, who was connected for two years with the construction of the original structures, will say a few words about that work which may be of interest.

The construction of the Metropolitan Elevated Railway of New York—known originally as the Gilbert Elevated Railroad—was begun in the fall of 1877. The contracts for building this road from Rector Street to 42d Street, on Church Street, West Broadway, South Fifth Avenue, and Sixth Avenue, were let, about November 1st, 1877, to the Phœnixville Bridge Company, the Keystone Bridge Company, and the Edgemoor Bridge Company.

Each of these companies agreed to complete its part of the railroad within 4 months, and to pay a penalty of $1 000 per day for any additional time that might be required.

As a matter of fact, the railroad was built from Rector Street to 42d Street—a distance of about 3 miles—in 4½ months. On account of the difficulties encountered, the penalty for the 2 weeks of extra time was not enforced. Considering the fact that the iron had to be rolled in Pennsylvania, and brought to New York in winter, the speed in construction was remarkable, and could scarcely be surpassed at the present time.

* New York City.

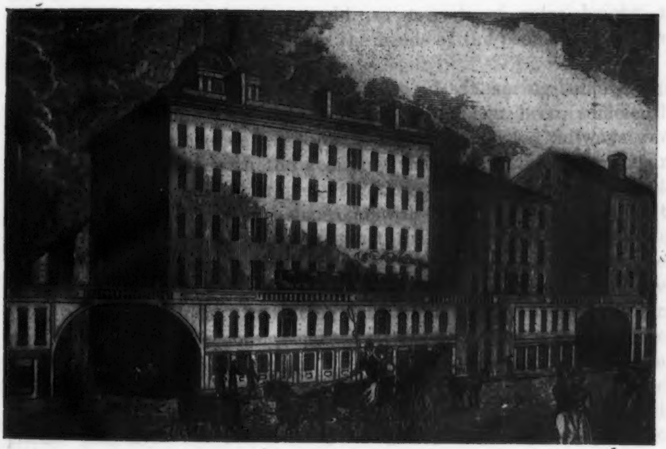


Fig. 135.—Proposed Plan for Elevated Railroad in New York City in 1833.

Mr. Wegmann.

The piers for the columns to support the structure were located from a base-line established on the west sidewalk, about 2 ft. from the curb. This line was measured with the utmost accuracy with a 25-ft., trussed, wooden, measuring rod, the length of which was compared from time to time with a standard rod, furnished by the bridge companies.

It was very difficult to establish a straight base-line on the crowded city sidewalks, especially on Sixth Avenue near 23d Street, which was then the great shopping district of New York. The speaker tried to run the base-line in the early morning, but found the weather too foggy, and finally resorted to the "pusher system". A member of the engineer corps or a laborer was stationed on each block. At a given signal, these men would push the crowd to one side, with loud shouts, and before the people had time to discover what caused the commotion, the line had been sighted. If the speaker had to do this troublesome kind of surveying again, he would use tripods of extra height, to enable the transitman to sight over the heads of the people by standing on a box.

As regards the wooden measuring rod, a 50-ft. spring-balance tape would have expedited the work, and would have been sufficiently accurate. The engineers engaged on those early elevated railways were very much impressed with the accuracy that was required, to insure that the ironwork would fit properly together. To show that this anxiety was somewhat unnecessary, the speaker will mention what happened to one column of the Sixth Avenue structure. In setting the templates for the anchor-bolts of the piers, the engineers measured, with a wooden rod, 23.5 ft. from the base-line to the center of the west pier, and then 23 ft. to the east pier. In setting the template for one of the piers, the engineer used the wrong mark on the measuring rod and placed the pier 6 in. too far east. When the ironwork was erected, the column on this pier was riveted to the cross-girder, and neither the workmen nor the engineer found out for some time that it was 6 in. out of plumb. The error was corrected by using a special base casting, which brought the center of the column into its required position.

The piers supporting the columns were built of brick, founded on large stones to which the anchor-bolts were fastened. The ordinary depth of each foundation was 10 ft., but, at the street intersections, where columns had to be built, according to the charter, the foundation depth was usually about 30 ft., in order to get down to the bottom of the street sewer, over which arches were generally built. Water and gas mains, etc., were usually straddled with I-beams, well protected by cement mortar.

On Sixth Avenue about ten piers were built daily on each section of the work. On the Second Avenue line, erected about a year later, about fifteen piers per day were built on each section. It required quick work on the part of the engineers, to give line and grade for the piers and to keep memoranda of how all the obstructions were bridged

Mr. Wegmann.

over. Very frequently the special plans for the piers, made at headquarters, arrived after the piers had been built, and all that the field engineer could do was to certify that the plans looked very much like the piers actually built.

The contract for the Sixth Avenue Elevated Railway was let by the Metropolitan Elevated Railway to the New York Loan and Improvement Company at $1 000 000 per mile. As engineer for the Keystone Bridge Company, the speaker knew the actual cost of the Sixth Avenue structures, which amounted only to about $500 000 per mile, including stations and track.

The Third Avenue Elevated Railway—which was a lighter structure than that on Sixth Avenue—cost only $325 000 per mile, according to figures published in technical papers at the time.

Mr. Constable.

HOWARD CONSTABLE,* M. AM. SOC. C. E.—Mr. Pegram has spoken of closed columns as possibly being worthy of attention. In 1883 the speaker made, for the late Octave Chanute, Past-President, Am. Soc. C. E., a report on the Interior Rusting of Wrought-Iron Columns. Many structures, in the East and Middle West, from 10 to 40 years old, were visited. A concave mirror with a small hole in the center was used to throw light up into the interior of a column, if open at the bottom, or through a 1½ or 2-in. hole, specially cut out, as near connections as possible.

In the case of the Omaha Bridge, there were a great many open columns, the interior of which could be easily inspected for a distance of about 6 ft. when the speaker was suspended from the lower chord of the bridge. The paint was mostly intact, and there were only occasional flakes of rust. In the case of a double webbed girder at Phœnixville, Pa., which was brought from England about 1840, the red lead paint in the interior was still slightly soft. No case was found where the interior rusting exceeded 0.5% of the cross-section. Some heavy rusting was found where columns were set in a cup-like shoe or base, also where lattice straps on the outside were not in full contact and left a capillary space for the retention of moisture.

The observation openings were made by first drilling a ⅜-in. hole, then using a special tool, somewhat like a washer cutter. The washer or disk with its hole, was kept as a sample, and the hole in the column was tapped and a plug screwed into place.

The speaker was so well satisfied that interior rusting was not as serious as had been claimed, that in later years he designed and erected a 6-story factory in which compressed air was taken up all the columns and used as required for smelting, sand blast, etc., by removing screw plugs in the columns and attaching pipes. To be sure, these columns were of cast iron, but their use as air ducts represented the passage of an immense quantity of air, more or less saturated with moisture owing to being under compression.

* Kingston, Mass.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

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Paper No. 1414

A BRIEF REVIEW OF TRIGONOMETRICAL MATHEMATICAL TABLES, AND A CONTEMPLATION OF THE SPECIFICATIONS FOR TRIGONOMETRICAL TABLES FOR GENERAL USE

By Virgil A. Eberly, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. William G. Raymond, Horace Andrews, George A. Christensen, and Virgil A. Eberly.

The writer has felt the need of tables of natural trigonometric functions which cannot now be obtained on the market, and takes this opportunity of explaining this need, which, if generally concurred in, may influence the prospective author to publish tables conforming to the specifications following.

A review of mathematical tables was made by the British Association for the Advancement of Science in 1873, and was very complete. The writer has recently searched the United States Congressional Library and the Library of the Naval Observatory, at Washington, D. C., the latter being exceptionally rich in its collection of mathematical tables. A great many of the tables described in the 1873 report may be seen there, as well as many abridged tables for various purposes, multiplication and conversion tables, and trigonometric tables produced since 1873. A few trigonometric canons produced since 1873 stand out prominently on account of the evident labor expended in

compiling them, and on account of the departure from what might be called established practice. The authors of these are:

(*a*). H. Andoyer (1911).
(*b*). J. Bauschinger and J. Peters (1910-11).
(*c*). C. Bremiker (1887).
(*d*). Chambers' Mathematical Tables, by J. Pryde (1899).
(*e*). Mrs. Emma Gifford (1914).
(*f*). J. Peters (1911).
(*g*). G. Rheticus, Opus Palatinum, by W. Jordan (1913).
(*h*). W. G. Raymond (1915).
(*i*). R. Shortrede (1903).

As compared with the older and fundamental tables, these, of course, have much better typographical arrangement and clearness, but it is interesting to note that some of them are lacking, as differences for tabular results are not listed as extensively as in the celebrated "Magnus Canon" (1596-1607), of G. Joachim Rheticus, the greatest of all the table computers, to whom is also due the canon of sines by Pitiscus (1613). The latter are rare, as indicated by a price of £36 15s. and £21, respectively.

Other tables with which American engineers may be more familiar are: Vega's logarithmic tables; Bruhn's logarithmic tables; Hutton's tables of logarithms of Numbers 1 to 108 000, and tables of logarithmic and natural trigonometric functions, etc.; Searles and Nagle's handbooks; etc.

Hutton's and Pryde's productions are very valuable, as they contain several tables which are not found in the books which are more prevalent at the present day. In the 1822 edition of Hutton may be found a very valuable bibliography of tables, if the reader cares to search for information relative to this subject. Other sources of information are: Glaischer's article on "Mathematical Tables" in the 11th Edition of the Encyclopædia Britannica; and William Wesley and Son's Natural History and Scientific Book Circular No. 147.

The writer submits the following propositions which should govern in the future preparation of trigonometric canons:

Proposition I.—Ninety degrees to the quadrant is and has been used so generally that it is necessary to adhere to this use.

Proposition II.—The degree should be decimally divided. Decimal division of the degree is the best for rapid computation, especially where the computations are extended, as in triangulation work. Arguments for such use may be seen in the preface of Professor W. G. Raymond's "Field Manual." In addition to these arguments, the following may be stated: it is easier (*a*) to secure values where interpolation is necessary; (*b*) to average a set of readings of angles; (*c*) to write an expression for an angle; (*d*) to avoid confusion with the abbreviations for feet and inches; (*e*) to plot curves on decimally divided curve sheets; etc.

Proposition III.—Differences should be listed in a column adjacent to all values given, preferably set half way between horizontal lines, in smaller and somewhat lighter type.

Proposition IV.—The tables should be seven-place. Computations should be carried out one place farther than field measurements, and, consequently, seven places is not too great a refinement for steelwork, triangulation, etc. Besides, if one uses a computing machine, seven-place functions are as easily manipulated as five-place.

Proposition V.—The proposed book would be one of natural functions only. Owing to the fact that no engineer who wishes to accomplish computations speedily will attempt to work without a computing machine, logarithmic functions should not be tabulated, as they are more cumbersome than natural functions when using a computing machine. In addition, natural functions have the advantage that actual figures for each partial computation are had, and an observation check as to the relative size of the part of the triangle being solved is readily made.

Proposition VI.—The book should be thumb-indexed. Page numbers should be omitted, as they are apt to be confused with degree numbers. Each page should contain 1°, with arguments to 0.01, except as noted under the following proposition.

Proposition VII.—The book should have four canons, as follows:

Canon I.—(Semi-quadrantal arrangement, 0° to 44° at top of page, 45° to 89° at bottom of page.)

Sine—Difference—Coversine (difference serving for both);

Chord—Difference;

Co-chord—Difference;

Versine—Difference—Cosine (difference serving for both).

Canon II.—(Quadrantal arrangement, 0° to 89° at top of page for tangent and secant, 0° to 89° at bottom of page for cotangent and cosecant.)

Tangent (cotangent same tabular value)—Difference;

Secant (cosecant same tabular value)—Difference.

Canon III.—(Quadrantal arrangement, 0° to 89° at top of page.)

Chord of the supplement—Difference.

Canon IV.—(Biquadrantal arrangement, 0° to 179° at top of page.)

Arc in terms of radius—Difference;

Arc in terms of circumference—Difference;

Area of segment in terms of circumscribing square—Difference.

Canon II should have arguments to 0.001° between 84° and 90°. Where the versine approaches zero, the cosine approaches unity, and the secant (also cosecant, as it is the same tabular value) approaches unity, values should be listed as indicated by the notation used in Merriman's "Civil Engineers' Pocketbook," page 1295, thus: $0.0_3 1259$ indicates that the 0_3 is to be replaced by three ciphers. As usually tabulated, these values are not comparable with other values listed in a seven-place table, because of the succession of similar figures.

***Proposition VIII.*—As indicated previously, there would be required twelve columns in Canon I (including the arguments at each side of the page), which would not be exceeded by any of the other canons, and, therefore, would govern in the size of page required. For an office book, therefore, a relatively large book would be necessary, in order to tabulate twelve columns for one hundred arguments. If printed on heavy paper, in large type, such a book would have the advantage, however, of remaining open when laid flat on a desk or table.**

In such a book of tables as outlined herein, there would be approximately 294 000 printed values, of which 65 334 would be arguments, and 229 656 would be tabular values which would have to be computed.

With about 1 200 printed values per page, the book would require about 246 pages. If, however, each page showed a separate degree

(except for Canon II, 84° to 90°), a book of about 459 pages would be required. The book should be provided with an explanation of the use of the tables with the use of the various computing machines; and especially of the values tabulated under "Area of segment in terms of the circumscribing square."

A page or two, as necessary, might be devoted to a conversion table from minutes and seconds to decimals of a degree.

The writer submits this paper with the belief that such a book of tables is not available, even in the usual form of degrees and sixtieths (minutes), and that engineers will gradually see the merits of the decimal division of the degree, just as they have seen the merits of the decimal division of the foot.

DISCUSSION

Mr. Raymond.

WILLIAM G. RAYMOND,* M. AM. SOC. C. E. (by letter).—Without doubt, the book proposed by Mr. Eberly would be of value to a certain class of computers, but it is thought that it would be altogether too large for general use. It is proposed to have one degree on a page, with the degree divided decimally. This would require a hundred lines, besides the headings, and would mean a page running from about 14 to 17 or 18 in. long. The book, therefore, would be very unwieldy, and, of course, could not be used at all for field operations.

The writer is a champion of the decimal division of the degree, being the author of the "Field Manual" referred to by Mr. Eberly, and is also a champion of five-place tables for field use, but realizes that seven-place tables are absolutely necessary for certain office computations, although not for the office solution of field problems. Nor is it practicable to use a computing machine for field calculations. Therefore, there must be two classes of tables. Indeed, perhaps, there should be three classes: one set for the great bulk of engineering computations that require not more than four-place tables; these should be of natural functions for use where computing machines are available, but of logarithmic functions where computing machines are not available; and until computing machines are very much reduced in cost, they will not be generally available for school classes or for offices having only a moderate quantity of computing work. The second class should be five-place tables for field use, and, for this purpose, as computing machines are not available, the tables should be logarithmic. It has been shown that the labor of computation is increased about 50% by the use of five-place instead of four-place tables, and about one-third by the use of six-place instead of five-place tables; and no ordinary field work is done with such precision as to warrant results certain to more than four significant figures, for which five-place tables give sufficiently exact results. Certain city, bridge, and geodetic surveys are made with greater precision, and require more extensive tables. The seeming precision of six-place over five-place tables is, for practically all ordinary field purposes, merely a seeming, but it appears to require great strength of mind on the part of engineers to realize this sufficiently to conform their practice to this truth. In the writer's judgment, there is really little excuse for six-place tables, five-place tables being all-sufficient for ordinary field operations and seven-place tables being required for work of higher precision, comparatively little work of an intermediate degree of precision being done. Therefore, the third class of tables should be seven-place.

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* Iowa City, Iowa.

Mr. Raymond.

Objection has been made to the use of the decimal division of the degree on the ground that it will cause the same confusion that is imagined by the objectors to the introduction of the metric system; but this is not true, because there is no change of the unit. The unit is the degree, or one-ninetieth of a quadrant, and as American surveyors and engineers have long since adopted for practically all field work the decimal division of the linear unit, there would seem to be no logical reason for not adopting the decimal division of the angle unit. The adoption of the decimal division of the degree would not lessen the value, or interfere with the ready reading, of any old records, but would materially lessen field work and the chances of error in passing from decimal to sexagesimal fractions, an operation that occurs in almost every field calculation involving angles.

Summarizing, the writer supports Mr. Eberly in his first and second propositions; namely, that there should be 90° to the quadrant, and that the degrees should be divided decimally; with his fourth proposition, where tables are to be used for office computation of high precision work; with his fifth proposition that the tables should be of natural functions only in case computing machines are to be used; and, in part, with his sixth proposition, that the book should be thumb-indexed and page numbers omitted; but it is suggested that each page should contain but one-half degree when the arguments are to one one-hundredth of a degree, in spite of the fact that this would double the number of pages for the angle tables. Proposition VII the writer does not feel competent to discuss. The canons included should cover the needs of all computers of high-precision work, but possibly the book should be arranged so that it could be published in parts, not all computers having need of all the functions indicated. Perhaps it seems a bit curious, but the larger the page the larger should be the type, that is, the greater the number of arguments on a page the larger should be the type, so that placing 100 arguments on a page instead of the usual 60 (or 50 for ½° with decimal division) will increase the size of the page more than in proportion to the number of arguments.

Mr. Andrews.

HORACE ANDREWS,* M. AM. SOC. C. E. (by letter).—The author indicates the need of tables such as those mentioned under his Sub-heading *g*, which were left incomplete by the death of Professor Jordan, but which, in all probability, would have been completed, as he had planned, under more favorable conditions for such work. The author designs to have the computing machine supplant the usual tables of logarithms of numbers, and this will reduce materially the size of his proposed tables.

* Albany, N. Y.

Mr. Andrews.

Referring to Mr. Eberly's numbered propositions, the writer would make the following remarks and suggestions:

Proposition I.—The use of the 90° division to the quadrant cannot be regarded as quite general, though it is nearly so. It may be expedient to adhere to this usage, but the absolute necessity for doing so is not apparent. There are many tables which are based on the division of the quadrant into 100 parts, or centesimal degrees—frequently indicated by a small letter *g*. These tables, for the most part, are from French, German, and Italian sources. Instrument makers in Europe quite generally advertise their readiness to furnish circles with the 100° division to the quadrant. It should not be forgotten that the metrical system is based on a division of the earth's meridian quadrant into 100 centesimal degrees, each of the length of 100 km. Where the metrical system is in use, it is logical also to adopt the centesimal degree. In certain classes of work, the centesimal division has not only all the advantages pertaining to the decimally divided degree, enumerated by the author in his second proposition, but others arising from a quadrant of 100 parts, and a semicircle of 200 parts instead of 180 degrees. Professor Jordan* carries out a complete traverse computation in duplicate, with old and new divisions of the quadrant, and remarks that: "With the continued changing by $\pm$ 180° one is liable to three times as many errors of computation as with the continued changing by $\pm$ 200^g." The convenience of having angles in the different quadrants easily located, as they are with the centesimal degree division, is at once apparent.

Proposition II.—The decimal division of the degree seems to have been proposed originally by Henry Briggs, and was used in his "Trigonometria Britannica" published in 1633, after his death; but this division never seems to have had any extended use. It was revived in Bremiker's five-place tables, in 1872, apparently as a protest against the centesimal degree, for Bremiker remarks in his preface that both systems are arbitrary, and indicates that a preference might be had for a division of the entire circle into 1 000 parts as more logical, though he confesses that it might not be easy to have the division of the day into decimal hours adopted. The writer agrees with the author that the decimal division of the 90° to the quadrant has attractions, but, many years ago, when he endeavored to make use of Bremiker's tables, he found the difficulties too great in the changes continually made necessary in using other tables, namely, the Ephemeris, where the sexagesimal division must be used first and then a conversion made into the decimal.

The writer is of the opinion that the change from sexagesimal to decimal division of the degree will be more difficult than the introduc-

* "Vermessungskünde", 2d ed., 1877.

Mr. Andrews.

tion of the metric system of weights and measures, and if at any time the latter is to be adopted, it would be more easy to use the centesimal degree than it would be to adopt a decimally divided degree of one-ninetieth part of the quadrant.

Proposition III.—The author does not indicate clearly the process of interpolation that he prefers, that is, whether he would use the small marginal tables of proportional parts, usual in tables of logarithms, or whether he prefers to give only differences and rely on the slide-rule for working out the proportion, which is the process favored by Jordan, and may be more easily and quickly applied to the larger differences existing in seven-place tables.

Proposition IV.—This may be open to question. Eight-place tables are now required for mathematical exactness in least-square adjustment of primary triangulation or other refined geodetic work; but, for the work usual in engineering, even where a very high degree of precision is sought, six places are almost always more than sufficient, and furnish a margin of precision sufficiently greater than that of the field measurements. As Bremiker points out, in his six-place tables, the numerical work of interpolation between tabular values is much abbreviated when the number of decimals used is reduced, the closeness between the arguments remaining the same as in the seven-place tables. For almost all work of an engineering nature, five-place tables are quite sufficiently precise. The degree of precision thus attainable is indicated in some of the editions of Gauss' five-place tables, which seem to have passed through a greater number of editions than any having a greater number of decimals. Any engineer having doubts as to the sufficiency of five places of decimals should carry out a few duplicate computations, using five places and seven places, and compare his results with the precision of his field measurements. If seven places should be chosen, as the author prefers, it would be very advisable to have editions with six places also available, on account of the greater ease of interpolation with the six-place table.

Proposition VI.—Thumb-indexing may be difficult of application to some tables where only a few pages are necessary; they also somewhat jeopardize the edges of a much-used book. Perhaps colored paper could be used to advantage—at least in some of the proposed tables—to facilitate their use, if for no other reason. It will be recalled that Babbage advocated the use of colored paper on other grounds; he considered that black on a white ground was a fatiguing combination for the eye, but it does not appear that any definite conclusion as to this was reached through the use of his tables on paper tinted yellow, brown, green, etc. Professor Gillespie, in his "Treatise on Land Surveying" (1855), had his tables printed on a yellowish-brown paper, following Babbage's principle. Merely for convenience in turning to a certain portion of a book, the use of colored paper, as is well known, is fre-

Mr. Andrews. quently adopted. This expedient would not seem to lessen the usefulness of a book of tables, and, if Babbage is correct, it might prove of value. Colored edges to tables were used in the "Tables de Logarithmes," Paris, 1868, of J. Dupuis, red edges being used on the tables of the circular functions.

Proposition VIII.—The mechanical execution of a book of tables subject to continuous and severe use is a matter of great importance. The book projected by the author might be furnished in paper covers and unstitched, as in many foreign books, so that those purchasers who wished to do so could have a strong hand-binding in place of the usual poorly attached "cases." The writer would suggest hand-sewing, all along, on five bands or tapes—flexible sewing—on the outside of the sections; all bands to be strongly attached to the covers and the whole covered with a good grade of buckram or other durable fabric, in place of the inferior leathers generally used commercially. The thinness of sections in the folio make-up advocated by the author would also be conducive to flexibility. There is no reason for not adopting the same manner of folding sections in an octavo, if thought advisable.

As there is little likelihood at present of any change from the sexagesimal division of the degree, it would seem prudent to adhere to that method of division if the proposed tables are to have any extended use.

Mr. Christensen. George A. Christensen,* Assoc. M. Am. Soc. C. E. (by letter).—The writer believes that a well-arranged set of tables, in convenient book form, and in accordance with the lines proposed, would be of great value to the Profession. It is generally realized, of course, that to edit such a book would be a work of love, and not one for profit. Mr. Raymond's comments have added practical specifications to the premises, and should be observed in case such a book of tables is prepared.

Attention is invited to Fig. 1 reproduced from a photograph showing an arrangement of multiplication tables, which could be applied to any table of natural or logarithmic functions. The writer has experimented with logarithmic tables arranged after this form, and has found that they could be used more expeditiously than the conventional form. This is true, also, of tables of natural functions, or of any other mathematical tables capable of being arranged in groups, as shown in Fig. 1. The advantage lies in the logical arrangement, the prominence of the index number, and that all numbers—for example, 1 to 100—are found on the two pages where the book opens. All functions pertaining to the index number with its decimals are located by arguments which are decimal co-ordinates, so to speak, of the functions sought. In this way any desired number is very readily found. The ten indexes are divided into two parts, forming the left-hand and right-hand pages of the book wherever it is opened.

* San Diego, Cal.

FIG. 1.—ARRANGEMENT OF A BOOK OF TABLES.

Mr. Christensen.

Attention is called to the form of index cutting of the book shown by Fig. 1. By actual trial it is found that the book can be rapidly leafed through, and any desired page quickly found by using only one hand. Another novelty is that the book is left-handed; it is operated with the left hand, the right hand being free to use the pencil or pen to make notations; it is truly a one-handed, left-handed affair. As an office appliance, it makes for efficiency, and, if a book of mathematical tables, such as proposed by Mr. Eberly, is published, the writer would recommend that the form described herein be adopted. The book was compiled by the writer, and a copy may be found in the Engineering Societies Library.

Mr. Eberly.

VIRGIL A. EBERLY,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer is obliged to concede that the tables he proposes would not be useful to all classes of computers, as the title of the paper indicated. Nevertheless, for triangulation, precise surveying, or structural work involving large numbers of connected triangles, such tables would save much time where the computing machine is used. He wishes particularly to emphasize the fact that the need of such tables has been brought about by the use of computing machines.

Such machines require a decimal system unless they are used for addition and subtraction only, and, if their use is thus restricted, they lose much of their value; besides, special machines departing from the decimal system are costly and of limited use. The solution of triangles calls for a machine with decimal arrangement, capable of performing multiplication, division, and extraction of square root. The machine, in turn, calls for a decimal division of the unit.

Mr. E. V. Huntington, of Harvard University, has compiled and published four-place tables using a decimal division of the degree, and has five-place tables in course of preparation, which, together with the fact that Mr. Raymond has published similar tables, would show that they are no longer experimental. The tables published by Mr. Huntington and Mr. Raymond do not possess all the facilities for interpolation, nor, in the number of canons, are they as extensive as those proposed by the writer.

Answering the question as to the method of interpolation, raised by Mr. Andrews, the writer prefers only the differences between consecutive values, set half way between the values. These differences are absolutely essential to speed with the computing machine, and are all that are required, under any circumstances, with a machine.

The writer agrees with Mr. Raymond that five-place require less time to operate than seven-place tables, but this difference is much less when a computing machine is used. Besides, the writer finds that, with a seven-place canon, having differences between consecutive values listed, much greater speed can be obtained than with a five-place canon

* Washington, D. C.

Mr. Eberly. without such differences, when it is necessary to interpolate, as frequently occurs in the classes of work indicated previously. This refers, of course, to use with a computing machine.

The objection which Mr. Raymond urges, of having 100 arguments per page, involves that which may be urged against many books of tables, namely, that the required length is somewhat great. However, the writer believes that the advantage of having one degree per page would offset this disadvantage. The book would not have the disadvantage of many works, that is, too great thickness, as it would have less than 500 pages. If the work were given the same size of type as that used in the natural functions in "Mathematical Tables",* with the same groups of five arguments and size of page margins, a page 11¼ in. deep would be required. The writer cannot see that such type would be too small, as tables are generally used with a straight-edge, regardless of the size of type. The five-group is much preferable to the ten-group arrangement.

The writer intended to state in the paper that, in case thumb-indexing is used, it should be confined to the different canons only, and not to the various degrees under one canon, as the latter would be so numerous as to render it worthless. The writer believes that an arrangement such as suggested by Mr. Christensen could be devised which, if tough, heavy paper was used, would be preferable to ordinary thumb-indexing.

He does not see that the question of the number of degrees per quadrant affects the situation, but adheres to the preference for 90, because the latter will save much interpolation.

The tables which are available for the classes of work previously mentioned, are not well adapted to the computing machine, principally because they were compiled before the day of the latter. Machines are now available at a cost of $250 or $300, and render the computation of work at a speed undreamed of when the tables now in use were compiled. The saving in time and accuracy soon overbalances the cost of the machine.

Referring to the question of the number of places in the tables, it is desired to remind Mr. Andrews that a seven-place value is the largest that one can accurately carry in mind. This should be given due consideration in the compilation of a proposed work.

* Edited by James Pryde, and published by W. and R. Chambers, London, 1899.

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Paper No. 1415

A PHENOMENAL LAND SLIDE—SUPPLEMENT

By D. D. Clarke, M. Am. Soc. C. E.

With Discussion by Messrs. George L. Dillman and D. D. Clarke.

Synopsis.

In 1904 the writer prepared an account* of a land slide at Portland, Ore., about 30 acres in extent, which was first observed during 1894, while the construction of two of the City's distributing reservoirs was in progress, and resulted in putting these reservoirs out of service for a period of nearly 10 years.

The original paper brought the history of the slide down to the early part of 1904, and described in considerable detail the various steps taken to determine the cause of the movement, its rate of progress, and the boundaries of the moving ground, as well as the efforts made to retard the movement and counteract its effect.

The purpose of the present paper is to continue the history of the slide down to date; to describe the construction of additional drainage tunnels supplementing the original drainage system; and to note also the results of surveys made at regular dates during the intervening period, as well as to describe the character and extent of the reservoir

* "A Phenomenal Land Slide," *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 322.

repair work undertaken in 1904, and completed successfully since that date, thus restoring the reservoirs and permitting of their uninterrupted use during the past 12 years.

Historical Notes.

As a preliminary to the present paper, the following brief summary of the original paper is submitted: During 1894 the Water Board of the City of Portland built two small reservoirs, each having a capacity of about 16 000 000 or 17 000 000 gal., situated in a small ravine in the City Park, about 2 miles from the business center of the city, and designed to supply the West Side, or main business district.

A short time before these reservoirs were completed a movement of the adjacent hillside was detected, which, at first, was thought to be entirely local and of minor importance. The work of constructing the reservoirs was thereupon pushed to completion, but the reservoir basins had scarcely been filled before the real magnitude of the movement became apparent.

The reservoir basins, therefore, were emptied at once, and instrumental surveys were promptly commenced in order to determine the extent of the slide. These surveys have since been continued at regular intervals, and by their results and a series of test borings and open shafts subsequently made and observed for a period of years (33 wash-drill borings and 22 open-shaft excavations reaching to bed-rock), the dimensions of the moving ground were at length determined to be approximately 1 700 ft. from east to west, and 1 100 ft. from north to south along the reservoir front—an area of approximately 29¼ acres—the depth ranging from 46 to 112 ft., the average being 77.8 ft. The approximate volume was 3 400 000 cu. yd., and the approximate weight 4 600 000 tons.

The borings and open shafts revealed the presence of a thin seam of blue clay along the surface of the bed-rock, with numerous water pockets in immediate connection therewith, several of the underground water pockets having considerable volume. Two of the largest of these water pockets were drained with pumps (the total pumpage aggregating several million gallons) with a marked deterrent effect on the

COMPARISON OF MONTHLY RAINFALL WITH MOVEMENT OF SLIDE

From January, 1895, to December, 1916

MOVEMENT OF SLIDING LAND

movement of the slide, as indicated by the periodical instrumental surveys.

Comparisons of Weather Bureau records of precipitation with the monthly movement of the slide indicated a close relationship between the two—if it did not offer absolute proof that the rate of movement of the slide depended on the volume of the rainfall during any series of months.

After a study of all the observed conditions, it at length became clear to the Water Board Engineers and the experts called into consultation, that the probable remedy was the construction of a system of drainage tunnels along the surface of the bed-rock, and that these should be located so as to tap the underground reservoirs which had been developed by the borings and open wells.

In accordance with the decision then reached, a total of 2 507 lin. ft. of such drainage tunnels, with timber supports, was constructed between June, 1900, and December, 1901, at a total cost of $14 161.14, or an average cost of $5.65 per lin. ft. for materials and labor.

The results secured by the construction of these drains were considered very satisfactory, and for a time it appeared as if the slide problem had been fully solved. That this confidence was not entirely unwarranted will appear from a study of the diagram showing the monthly rate of movement as compared with the rainfall for the years, 1895 to 1903, inclusive. This is shown as Plate XXV of the original paper, and in Plate XX the data are reproduced and extended to cover the period which has elapsed since the surveys were commenced. This diagram shows the average movement, per quarter, at approximately 50 stations along the central portion of the slide from east to west.

The volume of drainage from the tunnels was carefully observed for the 2 years following their completion, and was found to range from 10 000 to 15 000 gal. per day in summer, and from 25 000 to 75 000 gal. per day in winter; and at the end of 2 years it was decided that the drains were doing effective work and that it would be safe to proceed at once with the work of reservoir repairs.

Accordingly, in December, 1903, the writer submitted to the Water Board a report on the condition of the drainage work, stating the reasons why it appeared to be entirely safe to begin at once the work of repairing the broken reservoirs, and also submitting a plan for a permanent drain inside the tunnels already constructed.

Following the presentation of this report, the Water Board authorized an appropriation of $100 000, for reservoir reconstruction, etc., divided as follows:

For tunnel drains	$16 000
Repairs to Reservoir No. 3	36 000
Repairs to Reservoir No. 4	32 000
Roadway west of Reservoir No. 4	16 000
Total	$100 000

The fourth item covered the construction of a driveway along the west side of Reservoir No. 4, connecting with existing roadways around Reservoir No. 3.

During March, 1904, immediately following the adoption of the plan for tunnel and reservoir repairs, it was noted that there had been an accelerated movement of the slide. This was reported to the Water Board by the writer, who called attention to the unusual rainfall during the preceding 4 months, amounting to 27% more than the average for the same period during the past 21 years.

In this report the writer stated his belief that the increase in the movement of the slide, observed since the winter rains began, was due to the excessive rainfall, and that additional drainage tunnels should be constructed in order to restore the equilibrium indicated by the surveys made during the preceding 2 years. The writer also outlined a number of branch tunnels which he thought should be constructed.

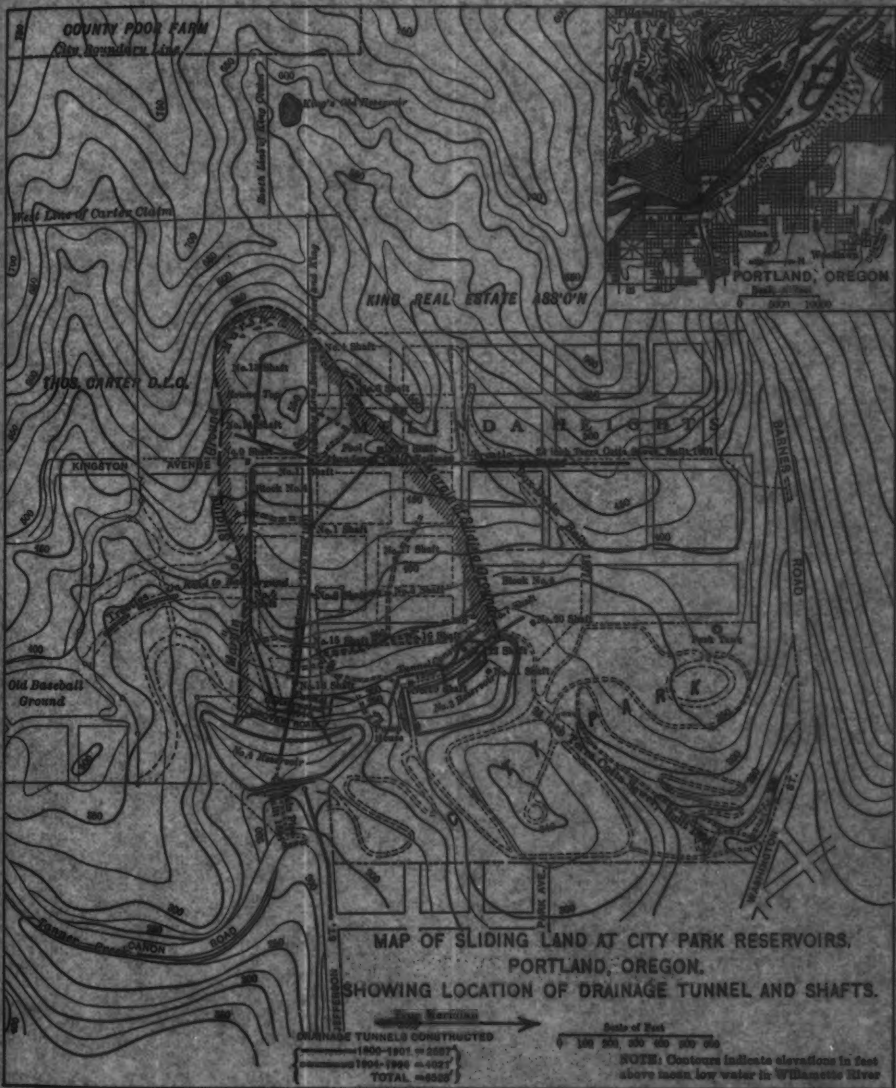
The Water Board immediately authorized the construction of these additional drains, and the work of completing them was carried forward in connection with the tunnel and reservoir repair work previously authorized.

As indicating the state of mind of the Water Board at this juncture, it is interesting to note that on March 29th, 1904, the day the foregoing tunnel extensions were authorized, the Board adopted the following resolution:

"In undertaking the repair of the reservoirs we feel the obligation to preserve the land lying west of them and to conserve the money so far expended on them, as far as we can.

"The complete system of drainage tunnels recommended by the Engineer apparently is the course to adopt.

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1415.
CLARKE ON
A PHENOMENAL LAND SLIDE.



"We feel, however, that we cannot be confident that the work when completed will be absolutely permanent and without possible mishap.

"Therefore we wish at this time, for the benefit of future committees, to note that there must be constantly exercised a most careful attention and a readiness and preparation for the possible expenditure at any time of a goodly amount for repairs."

In view of the foregoing the writer is pleased to note that only nominal repairs have been required at the City Park reservoirs since the completion of the tunnel and relining work, the total amounting to only a few hundred dollars for the 12-year period which has since elapsed, during which time the reservoirs have been in continuous service.

DRAINAGE TUNNEL EXTENSIONS.

The construction of the supplemental tunnel drains already noted was commenced early in the season of 1904, and was continued until 1906, when the system of drains was completed in accordance with the revised plans.

Plate XXI shows the location of these drains as related to those constructed several years earlier. These tunnels were mainly laid out from one to another of the several shafts excavated while the original exploration work was in progress.

In constructing the tunnels the excavated material was hauled from the tunnel heading to the nearest shaft in narrow-gauge cars, which were then hoisted to the surface and dumped.

The timbers used for tunnel supports, the cars and track, and the elevator cage used for this work were similar to those designed for the original tunnel project, as shown by Fig. 1.

Only a small force was employed on this work—one or two crews at different points, with sometimes two shifts per day, each crew consisting of:

One tunnel man,		$3.00	per day of 10 hours.
Two helpers,	each	$2.25	" " " " "
One or two top men	"	$2.25	" " " " "
One hoisting engineer.			

The timber supports were framed by a man especially detailed for that work.

The lumber cost from $10 to $12 per 1 000 ft. b. m., delivered at the shaft.

A total of 4 021 lin. ft. of new tunnel was built between April, 1904, and August, 1906, at a total cost of $26 896.20, exclusive of engineering and superintendence, or an average of $6.69 per lin. ft., as compared with $5.65 per lin. ft. for work of a similar character completed in 1900-01. This increase in cost was due largely to the advance in the prices of material and labor during the intervening period. In 1901 outside laborers were paid $2,00 per day of 10 hours, and tunnel men $2.25 and $3.00 per day; and timber cost $8.50 per 1 000 ft. b. m., delivered. In 1904 and 1905 the same rate per diem was paid for labor, but the working hours were reduced from 10 to 8. This is equivalent to an advance of 25% in the cost of labor; at the same time an equal or greater advance had taken place in the price of timber and other construction materials.

The 4 021 ft. of new tunnels added to the length originally constructed gives 6 528 lin. ft. in the complete drainage system.

The material encountered in the tunnel extension work was chiefly yellow clay, intermixed with fragments of basalt. No large pockets of water were discovered, and in that respect the work did not accomplish all that was anticipated, but the aggregate volume of drainage from all the branches has been large, ranging from 18 000 to 108 000 gal. per day during some years, the quantity depending on the season of the year and the attendant rainfall. During recent years the volume of this drainage has been somewhat less than noted above.

Concrete Tunnel Culvert.

It was realized from the beginning of the tunnel work in 1900 that the timber supports would soon decay, and that ultimately a more permanent construction would have to be adopted. With this end in view, a study was made to determine the best method of lining the tunnels so as to insure the permanency of the drains.

The design finally adopted for this work was that of a monolithic concrete sewer, 28 in. in diameter, to be built entirely inside the timber frames supporting the sides and roof of the tunnel.

This plan was a modification of one adopted for a sewer built at Truro, N. S., in 1902, Messrs. Lee and Coffin being the designing engineers.* The sewer at Truro was built in an open ditch, and the arch was of brick, the chief detail of interest being the removable centering.

* *Engineering Record*, August 30th, 1902.

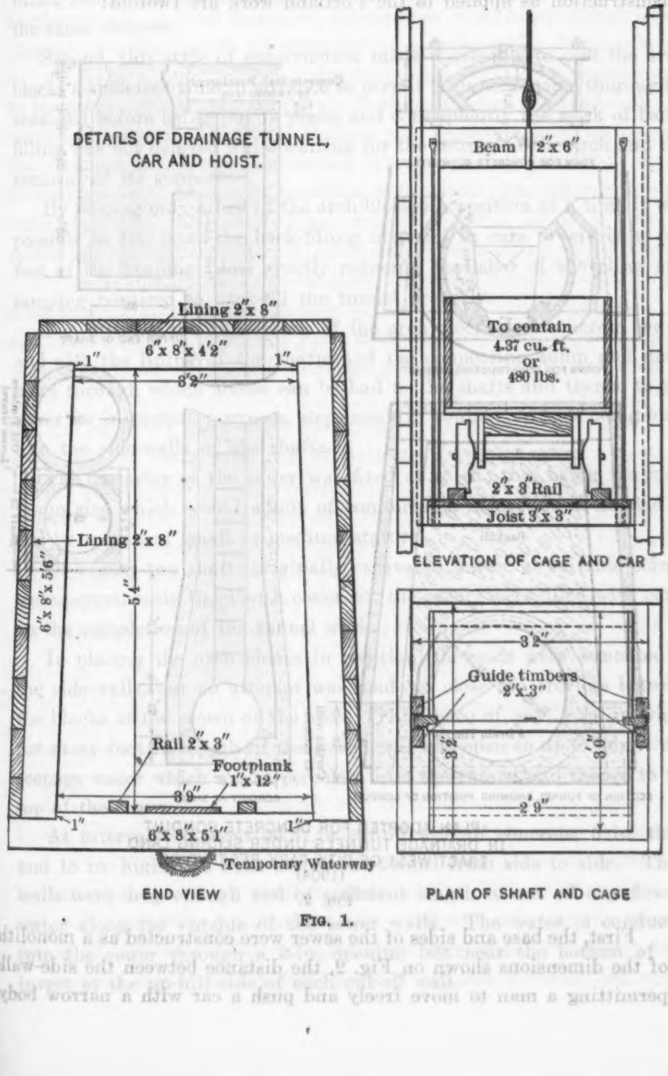

DETAILS OF DRAINAGE TUNNEL,
CAR AND HOIST.
Lining 2" x 8"
6" x 8" x 4'2"
1"
1"
3'2"
Lining 2" x 8"
6" x 8" x 5'6"
5'4"
Rail 2" x 3"
Footplank
1" x 12"
3'9"
1"
1"
6" x 8" x 5'1"
Temporary Waterway
END VIEW
Beam 2" x 6"
To contain
4.37 cu. ft.
or
480 lbs.
2" x 3" Rail
Joist 3" x 3"
ELEVATION OF CAGE AND CAR
3'2"
Guide timbers
2" x 3"
3'2"
3'0"
2'9"
PLAN OF SHAFT AND CAGE

FIG. 1.

For the Portland work the arch was made of reinforced concrete blocks, 12 in. in width and of sufficient length to span the opening between the side-walls. The advantages claimed for this method of construction as applied to the Portland work are twofold:

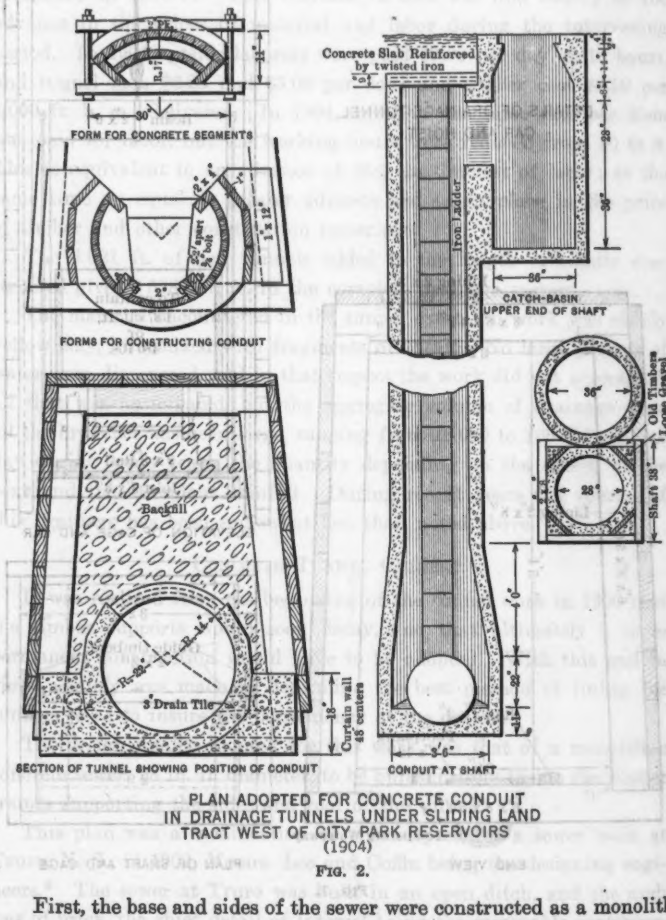


FIG. 2.

First, the base and sides of the sewer were constructed as a monolith, of the dimensions shown on Fig. 2, the distance between the side-walls permitting a man to move freely and push a car with a narrow body.

All construction materials were transported from the foot of the nearest shaft in cars running on a narrow-gauge track laid on the tunnel sills, or on the sewer invert after it had been built. The material for back-filling behind the side-walls and over the top of the sewer was hauled in the same manner.

Second, this style of construction made it possible to cast the arch blocks a sufficient time in advance to permit them to become thoroughly seasoned before being put in place, and consequently the work of back-filling was not delayed while waiting for the setting of the arch and the removal of its supports.

By placing only a few of the arch blocks in position at a time it was possible to transport the back-filling material in cars to within a few feet of the heading, thus greatly reducing the labor of shoveling and tamping required to back-fill the tunnel properly.

Fig. 2 shows the dimensions of the arch blocks and concrete sewer, and also the lining of the shafts and the connecting sump and man-holes through which access can be had to the shafts and thence to the sewer for inspection purposes, steps made of round iron rods being built into the side-walls of the shafts.

The diameter of the sewer was fixed at 28 in., that being the minimum size which would admit of comfortable inspection from end to end by a man of small or medium stature.

Of twenty-two shafts originally excavated, seven, at suitable points, were permanently lined with concrete; the others were filled with earth on the completion of the tunnel work.

In placing the arch blocks in position, the ends were cemented to the side-walls, but no attempt was made to close the crevices between the blocks at the crown of the arch. This space of, say, ¼ in. in width for every foot in length of the sewer, was left open so as to admit any seepage water which might percolate into the tunnel and thence to the top of the sewer.

At intervals of about 50 ft. a cut-off wall of concrete, 6 in. thick and 18 in. high, was built across the tunnel from side to side. These walls were deep enough and of sufficient length to cut off any flow of water along the outside of the sewer walls. The water is conducted into the sewer through a 3-in. opening left near the bottom of the invert at the up-hill side of each cut-off wall.

At each side of the sewer a line of 3-in. drain tile was laid, connecting with the opening into the sewer at intervals of 50 ft., this opening being a few inches above the upper face of the sewer invert.

The construction of the tunnel conduit was commenced in May, 1904, and the relining of the original project of 2 507 ft. was completed in May, 1905. The relining of the 4 021 lin. ft. of extension tunnels was commenced in 1908 and completed in 1910.

For the invert and side-walls of the tunnel, the concrete consisted of one part Portland cement, two parts Columbia River sand, and three parts of Willamette River gravel not exceeding 1 in. in diameter, all by volume; the arch blocks were composed of a mixture of one part of cement to three parts of concrete sand, mixed dry and thoroughly rammed. Two ¼-in. twisted iron rods, 12 in. long, curved to the radius of the mould, were embedded in each arch block.

The construction of the tunnel conduits was carried forward by a small crew of men in connection with the work of excavating new tunnels and the repair of the reservoir linings in progress during 1904.

The concrete materials used in the work were furnished by the contractor, who delivered materials of the same class for the reservoirs; the mixing and laying of the concrete were done by day's labor under the direction of the Department foreman. The tunnel foremen were paid $3.00 per day, and other inside labor $2.25 per day.

Detailed costs, kept during the period from June 1st, 1904, to June 1st, 1905, showed that 2 746 lin. ft. were completed at an average cost of $3.20 per lin. ft. for the materials and labor for constructing the conduit and back-filling the tunnel.

Repair of Reservoirs Nos. 3 and 4, in 1904.

One effect of the slide had been to shatter badly the concrete lining on the western slopes of both reservoirs, and the problem was to replace this lining, and also to reinforce the old lining on the bottom and on the eastern slopes of the reservoirs, so as to insure that the basins should be thoroughly water-tight.

As originally built, the lining of these reservoirs consisted of cement concrete placed on the bottom and sides of the basins, the sides having been graded to a uniform slope of about 1 to 1½. In places where the material forming the slopes was of a rocky character a

layer of clay and gravel puddle, 8 in. thick, had been placed under the concrete. For the purpose of supporting and anchoring the lining to the slopes, a network of ½-in. twisted iron rods was embedded in the concrete about midway between the upper and lower faces. These rods were about 20 in. apart, and, at intervals of about 10 ft., they were anchored to the tops of concrete posts, about 8 in. in diameter and 3 ft. long, moulded in place in the slopes.

A system of 4-in. tile drains under the western slopes also formed a part of the original plan. These drains were about 30 ft. apart, and extended up and down the slopes. The trenches in which they were placed were about 12 in. deep, and were filled with coarse sand after the tiling was laid.

During the few months the reservoirs were in use in 1895 it was demonstrated beyond question that the original linings were not as effective as they were designed to be. The case was a complicated one, for, in addition to the original thickness of the concrete and the underlying layer of puddle or compacted earth, the concrete was laid in sections with an expansion joint ½ in. wide running up and down the slopes at intervals of from 12 to 20 ft. For the lower half of these joints the concrete of adjoining sections was laid in close contact, but without attempting to form a thorough bond; the upper half of the joint was filled with asphalt or asphaltic mastic, the entire upper surface of the lining then being covered with a mop-coating of hot asphalt.

Considering the length of the slopes—from 50 to 80 ft.—and the extreme depth of the basins—41 ft. behind one dam and 49 ft. at the other—the problem of making repairs that would be effective was one that caused the writer no little anxious thought. Nowhere in the literature of engineering within his reach at the time could he find any account of work of a similar character, undertaken under conditions nearly as severe as existed here. These conditions may be enumerated as follows:

1.—The extreme pressure to which the linings would be subjected (maximum pressure, 22 lb. per sq. in.);

2.—The long slopes without a break; and

3.—The necessity for providing for a water-proofing course which of itself should possess sufficient strength and elasticity to span any small opening in the foundation layer of concrete,

which might be caused by a renewed or spasmodic movement of the slide.

The apparent necessity for relining the entire surface of each basin also added to the expense of the work.

Without describing in detail all the steps of the evolutionary process by which a conclusion was reached regarding the method to be adopted for making these repairs, it may be stated briefly that it was decided:

First.—That the original concrete lining on the west slopes of both reservoirs should be removed where broken or unsound and the banks dressed down; that the old tile drains should be cleaned out and additional under-drains constructed, the trenches to be filled with gravel and broken stone, instead of sand. (At one point on the west slope of Reservoir No. 4 the movement had reached a maximum of 3 ft., making necessary a re-grading of the slope for a considerable distance.)

Second.—That a new base course of concrete, having a thickness of 7 in. at the base of the slope and 5 in. at the top, should be placed on the slopes thus prepared.

Third.—That on top of the new layer of concrete, and on the top of the old lining covering the bottom and east slopes, there should be placed a layer of water-proofing material of sufficient strength to insure that it would bridge over any small cracks which might subsequently develop in the foundation course.

Fourth.—That on top of this water-proofing course there should be placed a second layer of concrete, 6 in. thick at the base of the slopes and 4 in. at the top; and that the supplemental layer on the bottom of the reservoir basin should have a uniform thickness of 5 in. At the foot of the slopes the concrete layers were to be made thicker, so as to round off the angle between the sides and the bottom. This is shown by the cross-section of No. 3 reservoir basin, Fig. 3. The rectangular footing, about 2 by 3 ft., shown on the plans, was only extended along the west slope of Reservoir No. 3.

In carrying forward the repairs of the reservoir basins, the following procedure was adopted:

The removal of the broken concrete lining on the west slopes of both reservoirs was done by Water Department forces, the material being piled at convenient points outside of the basin, and removed later and utilized for paving roads in the adjacent Park.

Contracts were entered into by the Department for purchasing and hauling lumber, cement, sand, gravel and crushed rock; also for asphalt and felt. Other supplies in smaller quantities were purchased on requisition.

Contracts were also made covering the labor of mixing and laying the concrete, and for laying the felt and asphalt water-proofing, materials being furnished by the City at the site of the work.

The prices paid for materials and labor required for the reservoir repairs were as follows:

Cement, \$2.40 to \$2.63 per bbl.

Broken stone, \$2.35 to \$2.42 per cu. yd.

Gravel, \$1.45 to \$1.55 per cu. yd.

Sand, \$1.15 to \$1.85 per cu. yd.

Felt, f. o. b. factory, \$1.25 per 100 sq. ft.

Asphalt, \$16.00 per ton.

Labor, laying concrete (all materials furnished):

Foundation and top course,	\$2.75 per cu. yd.
Cement finish,	\$0.12 per sq. yd.
Parapet wall,	\$3.00 per cu. yd.
Laying felt: first layer,	\$0.35 per 100 sq. ft.
second and third layers,	\$0.25 " " "
Hauling cement,	\$0.095 per bbl.

The following extract from the specifications for the work shows the proportions adopted for the concrete mixture:

"In proportioning materials for mortar, grout, and concrete, the volume of cement shall be taken to mean 380 lb. net. One volume of sand, gravel or stone shall be taken to mean 3.7 cu. ft. loosed or shaken down. Sand, gravel and broken stone shall be measured in barrels or boxes. Measurement in wheel-barrows will not be permitted. The concrete for the foundation course on the west slope shall be proportioned as follows: 1 part cement, 3 parts sand, and 6 parts broken stone or gravel. For the surface layer of concrete the proportions shall be, 1 part cement, 2½ parts sand, and 4 parts gravel or stone, as the Engineer may direct."

It may be stated that broken stone was used in the concrete forming the base course on the west slope, resting on the earth, but for the upper layer, which rested on the felt water-proofing, gravel was substituted for broken stone in order to avoid the danger of puncturing

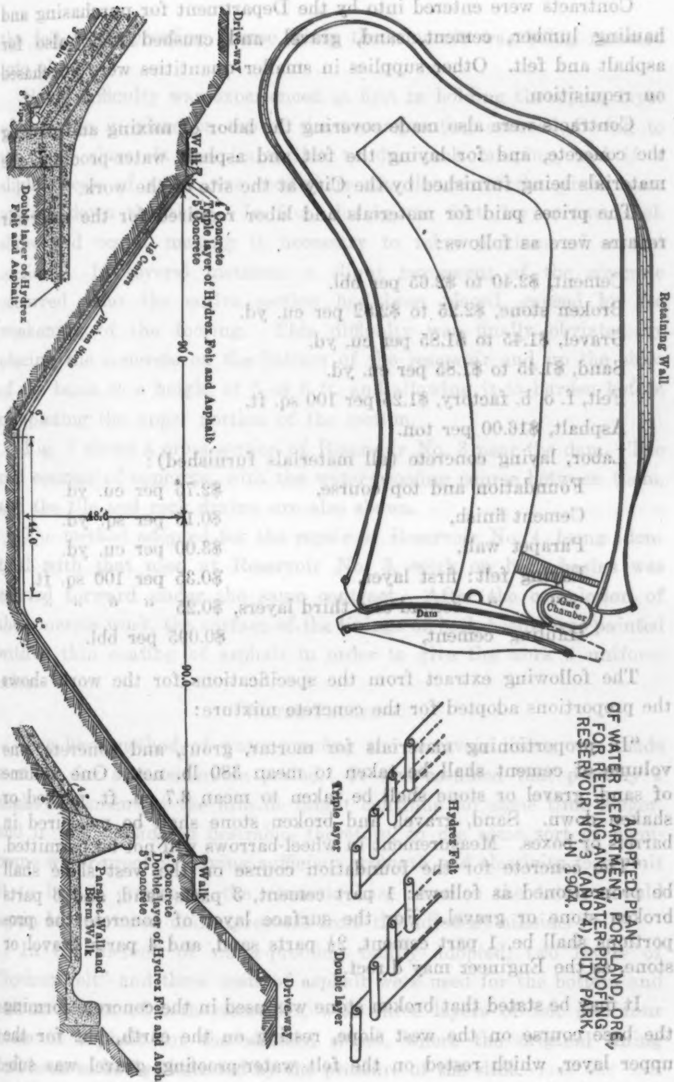


FIG. 3.

Contracts were entered into by the Department for purchasing and hauling lumber, cement, sand, gravel, and crushed rock, also for asphalt and felt. Other supplies in smaller quantities were purchased on requisition.

Contracts were also made covering the labor of mixing and placing the concrete, and for laying the felt and asphalt water-proofing, the materials being furnished by the City at the site of the work.

The prices paid for materials and labor required for the reservoir repairs were as follows:

Cement, $2.40 to $2.65 per bbl.
Broken stone, $2.25 to $2.82 per cu. yd.
Gravel, $1.45 to $1.85 per cu. yd.
Sand, $1.45 to $1.85 per cu. yd.
Felt, f. o. b. factory, $1.25 per 100 sq. ft.
Asphalt, $16.00 per ton.
Labor, laying concrete (all materials furnished):

Foundation and top course,	$2.75 per cu. yd.
Cement finish,	$0.15 per sq. yd.
Parapet wall,	$3.00 per cu. yd.
Laying felt: first layer,	$0.35 per 100 sq. ft.
" " second and third layers,	$0.25 " " " "
Hauling cement,	$0.095 per bbl.

The following extract from the specifications for the work shows the proportions adopted for the concrete mixture:

"In proportioning materials for mortar, grout, and concrete, one volume of cement shall be taken to mean 380 lb. net. One volume of sand, gravel or stone shall be taken to mean 3.7 cu. ft. packed or shaken down. Sand, gravel, and broken stone shall be measured in barrels or boxes. Measurement in wheel-barrows will not be permitted.

"The concrete for the foundation course on the west slope shall be proportioned as follows: 1 part cement, 3 parts sand, and 6 parts broken stone or gravel. For the surface layer of concrete the proportions shall be, 1 part cement, 2½ parts sand, and 4 parts gravel or stone, as the Engineer may direct."

It may be stated that broken stone was used in the concrete forming the base course on the west slope, resting on the earth, but for the upper layer, which rested on the felt water-proofing, gravel was substituted for broken stone in order to avoid the danger of puncturing

the felt water-proofing course while the concrete was being rammed into place.

Some difficulty was experienced at first in holding the upper layer of concrete in place on top of the water-proofing course. Owing to the steep slope, it was impossible to do much tamping, and if a slight excess of water was used there was danger of the whole mass moving down the slope. In fact, during the first day or two, such slides did occur, making it necessary to relay portions of several sections. In several instances a slight movement of the concrete occurred after the entire section had been placed, caused by the weakening of the footing. This difficulty was finally obviated by placing the concrete on the bottom of the reservoir and up the sides of the basin to a height of 5 or 6 ft. and allowing it to harden before completing the upper portion of the section.

Fig. 3 shows a cross-section of Reservoir No. 3 near the dam. The two courses of concrete, with the water-proofing course between them, and the tile and rock drains, are also shown.

The method adopted for the repair of Reservoir No. 4, being identical with that used at Reservoir No. 3, work on both basins was carried forward under the same contract. After the completion of the concrete work, the surface of the linings of both basins was painted with a thin coating of asphalt in order to give the work a uniform color.

Water-Proofing.

The best method of water-proofing the reservoir lining was made a study for a considerable period. It was assumed that possibly a slight movement of the hillside might continue for some time longer, and it was considered desirable, therefore, to use some sort of membrane water-proofing having sufficient strength and elasticity to permit it to bridge cracks in the concrete base in case the latter should again be fractured under pressure from the adjacent hillside.

In the method of water-proofing finally adopted, two layers of "hydrex felt" and three coats of asphalt were used for the bottom and eastern slopes of both reservoirs, with three layers of felt and four coats of asphalt for the western slopes, where the original lining had been so badly shattered by the pressure of the slide.

This water-proofing membrane method practically conformed to that adopted by the Pennsylvania Railroad engineers for water-proofing

the Hudson River Tunnel, then under construction, except as to the number of layers of felt, and the substitution of asphalt as a binder in place of coal-tar pitch specified for the Pennsylvania Railroad work. The change in the binder coat was made on the suggestion of the late Alfred Noble, Past-President, Am. Soc. C. E., Consulting Engineer for the Pennsylvania Railroad work, because it was thought asphalt would make the membrane more elastic and hence more suitable.

In order to demonstrate the effectiveness of a membrane lining similar to that just described, under conditions comparable with those under which it was to be used, it was decided to experiment with a typical section before the material was finally adopted. Accordingly, a small section of such a membrane, about 1 sq. yd., was built up for testing purposes. This consisted of two layers of felt, each coated with asphalt on both sides, which was subjected to a water pressure of 50 ft.—the maximum working pressure at Reservoir No. 3. The water chamber used for testing purposes, to which this pressure was applied, was of cast iron, except that one side consisted of fir planks 3 in. thick and 12 in. wide, with a space of ½ in. between adjacent planks. The section of membrane to be tested was placed on the inner face of these planks, and was supported by them, the membrane spanning the ½-in. opening between the planks. The joints around the margin of the chamber were sealed with a soft rubber packing.

During the test, which was continued for nearly 6 months, it was found that under pressure the membrane was forced into the cracks between the planking, and for a period of 146 days did not weaken sufficiently to allow the water to escape from the 6-in. stand-pipe supplying the pressure. It was concluded, therefore, that the material was of sufficient strength and elasticity to answer the purpose in view.

It may be said, further, regarding this test, that after the pressure had been maintained for 146 days it was released for 119 days, at the end of which time it was again applied and the test continued. At the end of a further period of 44 days, a slight crack, a few inches in length, developed in the felt where it had been compressed into the space between the planks, thus allowing the water to escape from the stand-pipe and release the pressure. The actual test, therefore, covered a period of 190 days.

FIG. 4.—RELINING EAST SLOPE, RESERVOIR NO. 3, SEPTEMBER 13TH, 1904.

FIG. 5.—RELINING EAST SLOPE, RESERVOIR NO. 3, SEPTEMBER 13TH, 1904.

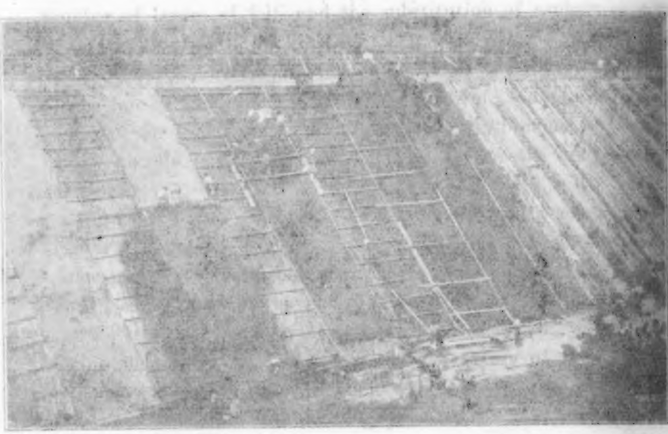


FIG. 4.—RELINING EAST SLOPE, RESERVOIR NO. 3, SEPTEMBER 13TH, 1904.

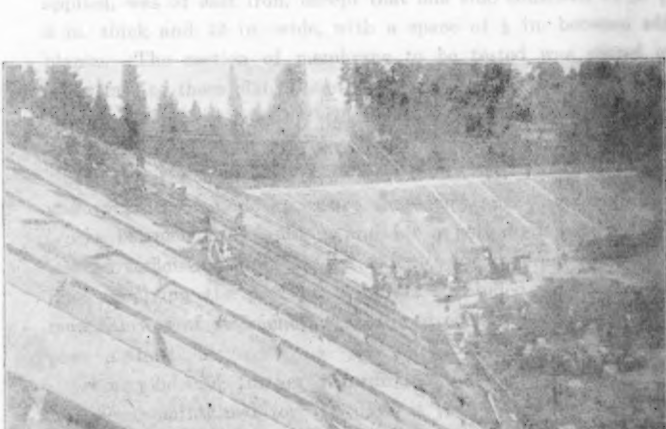


FIG. 5.—RELINING EAST SLOPE, RESERVOIR NO. 3, SEPTEMBER 13TH, 1904.

FIG. 6.—WEST SLOPE, RESERVOIR NO. 4, SHOWING UNDER-DRAINS, SEPTEMBER 21ST, 1904.

FIG. 7.—RELINING WEST SLOPE, RESERVOIR NO. 4, OCTOBER 17TH, 1904.

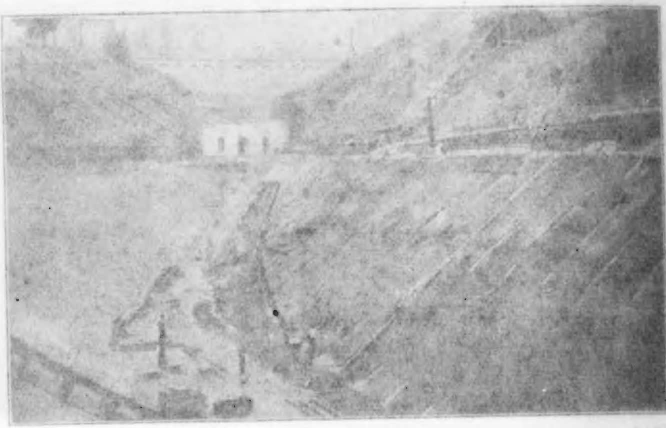


Fig. 6.—West Slope, Reservoir No. 4, Showing Under-Drains, September 21st, 1904.

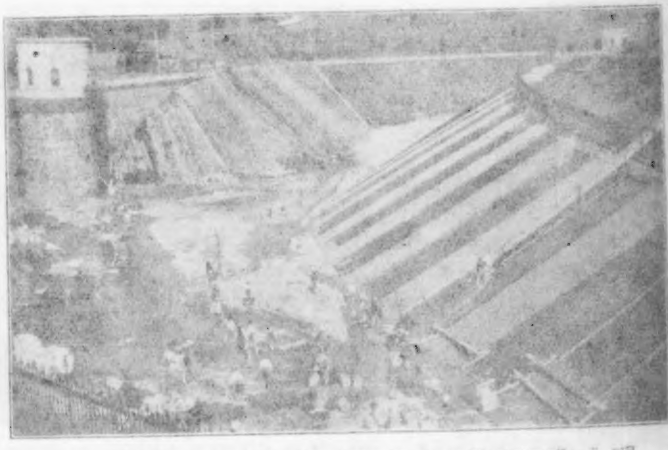


Fig. 7—Relining West Slope, Reservoir No. 4, October 17th, 1904.

FIG. 8.—RESERVOIR NO. 3: HORIZONTAL CRACKS IN BUTTRESS, AND BREAKS IN WEST SLOPE AND PARAPET, SEPTEMBER 28TH, 1897.

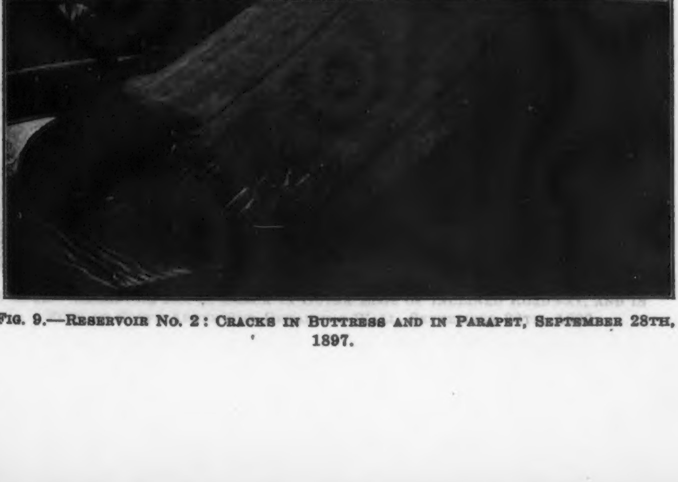


FIG. 9.—RESERVOIR NO. 2: CRACKS IN BUTTRESS AND IN PARAPET, SEPTEMBER 28TH, 1897.

Fig. 10.—Reservoir No. 4: Cracks in Parapet Wall and West Slope, September 28th, 1897.

Fig. 11.—Reservoir No. 4: Crack in Outer Edge of Inclined Roadway, and in Face of Slope Above Sub-Retaining Wall, September 26th, 1897.

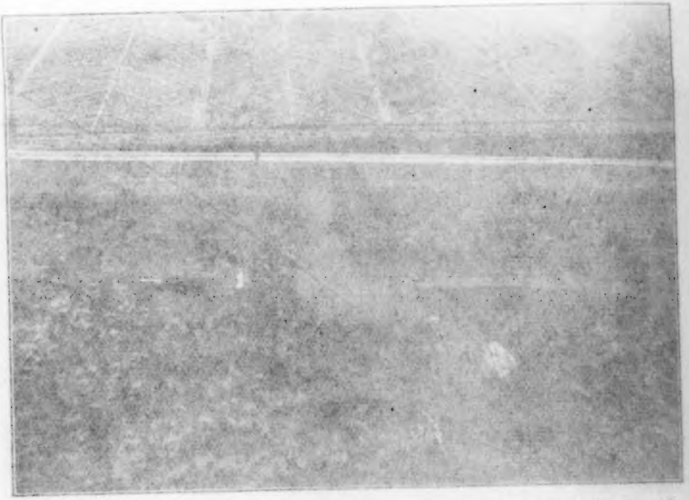


FIG. 10.—RESERVOIR NO. 4: CRACKS IN PARAPET WALL AND WEST SLOPE, SEPTEMBER 28TH, 1897.

FIG. 11.—RESERVOIR NO. 4: CRACK IN OUTER EDGE OF INCLINED ROADWAY, AND IN FACE OF SLOPE ABOVE SUB-RETAINING WALL, SEPTEMBER 28TH, 1897.

FIG. 12.—RESERVOIRS NOS. 3 AND 4, PORTLAND, ORE., BEFORE REPAIRS WERE COMPLETED IN 1904.

FIG. 12.—RESERVOIRS NOS. 3 AND 4, PORTLAND, ORE., BEFORE REPAIRS WERE COMPLETED IN 1904.

Fig. 13.—Reservoir No. 3 Since Repairs Were Completed in 1904.

Fig. 14.—Reservoir No. 4 Since Repairs Were Completed in 1904.

Fig. 13.—Reservoir No. 3 Since Repairs Were Completed in 1904.

Fig. 14.—Reservoir No. 4 Since Repairs Were Completed in 1904.

A small section of this membrane, showing the crack which developed under the foregoing test, was cut out at the conclusion of the work, and has since been preserved. A recent examination shows that at present this sample of the membrane lining has a considerable degree of flexibility, notwithstanding the fact that it has been stored in a dry and warm room for 12 years.

In laying the sheets of felt, they were lapped from 2 to 4 in. at their edges, and from 6 to 12 in. at their ends, there being three layers of felt for the west slopes and two layers for the remainder of both basins.

Regarding the effectiveness of the water-proofing work and the condition of the reservoirs at the present time, the following may be stated: When the basin of Reservoir No. 3 was first filled, in March, 1905, after the relining was completed, a slight seepage through the under-drain was observed when the water behind the dam was only about 10 ft. deep, the slope of the reservoir bottom being such that the flow line did not then extend more than about half the length of the basin. The system of drains under the lining centers in a single 6-in. pipe which passes through the base of the dam, the flow in this pipe being controlled by a gate in a chamber in front of the dam. The measured flow from this drain, when first observed, was only at the rate of about 8 000 gal. per day. As the water in the basin arose, the flow from the drain increased, due to the additional head. This has been the case every time the reservoir has been emptied and refilled; for several years the maximum flow when the reservoir was full has been about 36 000 gal. per day; and, for the last year or more, it has ranged from 55 000 to 60 000 gal. per day.

No serious attempt has been made to repair this leak, but it is thought to be near the dam, if not at the joint between the lining and the vertical face of the dam.

The loss of water is not a serious matter at present, and hence the question of further repairs to the lining has not been considered important. It can be said that this is the only defect in the work undertaken at Reservoir No. 3.

At Reservoir No. 4 no such leakage has occurred. At this basin, however, a slight cracking of the west parapet wall has been detected, and one or two of the fence posts are now inclined outward. A 6-in.

space was left between the top of the slope lining and the parapet wall, and this has not yet been closed by the earth pressure.

Seepage through the concrete dams has been in evidence for some years and, in consequence, the outer faces of both dams of Reservoirs Nos. 3 and 4 have become much discolored from the laitance deposits (as appears from an examination of Figs. 13 and 14), no attempt having been made to water-proof the inner or water face of the dam at either reservoir in connection with the relining of the slopes.

Figs. 4 to 7 show the workmen at Reservoirs Nos. 3 and 4 engaged in laying and coating the hydrex felt and mixing and placing the concrete covering at different stages of the work.

Figs. 8 to 14 show the reservoir linings before repairs were commenced and the basins and dams as they now appear.

During 1894, the late J. D. Schuyler, M. Am. Soc. C. E., was the Consulting Engineer in charge of the design and construction of the four reservoirs then being built by the City, including the City Park Reservoirs, Nos. 3 and 4, described herein.

The reclamation work described in the original paper was carried on under the direction of the late Isaac W. Smith, M. Am. Soc. C. E., who was Chief Engineer until his death on January 1st, 1897, the writer being Principal Assistant Engineer during this period. Since that date the writer has been Chief Engineer in charge of the work herein described.

DISCUSSION

GEORGE L. DILLMAN,* M. AM. SOC. C. E. (by letter).—It may be said, generally, that engineers have few opportunities to examine their works after completion. Mr. Clarke is to be congratulated on his opportunity to supplement the first drainage by the second construction. The Society is to be congratulated on this supplementary paper. Mr. Dillman.

Without doubt, the reservoir slide at Portland has had the most thorough investigation of any slide on earth. It has extended over many years, has included many shafts and wells through the slide, tunnels under and through it, and careful observation of its movement, in amount and direction. A lawsuit for damages developed facts showing that motion had been periodical for many years before it was recognized as a slide. The surface was covered with timber, and was uninhabited, so that the pertinent facts—cracks at the head of the slide into which cattle fell and had to be rescued, the irregular leaning of trees, and the springs at the toe of the slide now occupied by reservoirs—were not connected as symptoms of a single phenomenon.

The writer now believes that the slide described was in motion for years, possibly hundreds of them, prior to the coming of man, certainly prior to the first Government survey across it. He also confesses to carelessness in observation, when the cable railway, shown on Plate XXI, from Jefferson Street to the City Park, was built in 1891. The banks of the cuts slid, the slot closed, the cast yokes broke, the trees could not be felled by expert woodsmen in the direction desired, and men on the bridgework across what is now No. 4 Reservoir made a path up the gulch to about where No. 3 Dam was afterward built, to a fine permanent spring. There was a small stream down this gulch, following the toe of what is now known as the slide.

Others were just as unobservant, as is shown by the construction of the reservoirs before learning of this general slide. No one can truthfully say "I told you so" prior to the failure of the western slopes of those reservoirs. Such able engineers as the late James D. Schuyler, and Isaac W. Smith, Members, Am. Soc. C. E., and the author, ignored it. The late W. H. Kennedy, M. Am. Soc. C. E., then Chief Engineer of the Oregon Railroad and Navigation Company, stated to the writer, when the damage suit was on, that the excavation of the reservoirs undoubtedly caused the slide. He changed his mind when the evidence was seen to be overwhelming that the slide had periodic motion long prior thereto. Except Mr. Clarke, these engineers have gone, but they were all men of recognized ability.

To some little extent prior to 1891, but to a very great extent since, the writer has been studying slides. They are the most common phenomena of the Pacific Coast. He knows of them from the Straits of

* San Francisco, Cal.

Mr. Dillman.

Juan de Fuca to Panama, and has information of them in South America. At first, it would seem that slides are of different kinds; they move laterally, they upheave, they sink, and some seem to be dry. Some move fast enough to be called avalanches, some are slow and regular, and are called locally mud glaciers, some move only at long intervals.

When the Canadian Pacific Railway was built, settlement affected irrigation along it; in one place, a very large area started toward the river. That slide was arrested by stopping the irrigation. The Canadian Pacific Railway Company purchased the farms and abandoned them.

The Oregon Railroad and Navigation Company first built its road past the Cascades of the Columbia on a shelf blasted out of solid rock. The road still rests on the same shelf, but instrumental work has shown that it has moved (with its mountain) more than 17 ft. northward. That slide is slow. The depth of the river above shows that the Cascades, originally, were made by a slide. Probably erosion is compensated for by motion, certainly from the south, possibly from the north. The Indian legend of a natural bridge at the Cascades is entirely reasonable, and is supported by collateral facts.

There is a slide at Tillamook, which includes several hundred acres of land and the Tillamook Light. Its motion is small, and might not have been discovered but for the failure of a small bridge across its trace on the surface.

The Cow Creek slide on the Oregon and Cascades Railway, in Oregon, interrupted operations so many times and so seriously as to warrant several miles of new construction of heavy work in order to avoid it. This slide has never been cured, and is probably not worth curing.

A slide in Berkeley, Cal., a few years ago included many houses. It was cured (thus far completely) by the abandonment of a lily and frog pond at its head. Until the writer examined and pointed out this contributing cause, the pond had been maintained by a Professor in the Geological Department of the University of California. Geology has much to say about faults after they occur, but may not go into causes. Incidentally, all faults are caused by slides. Slides may be caused by earthquakes, but most of them are from something else.

All the railroads on the Pacific Coast have to contend with slides. The Northwestern Pacific has the finest collection along its Eel River line. Some are curing themselves, some are being cured by drainage—sub and surface. The Southern Pacific is affected at a few points in the Sacramento Canyon, at Carquinez Straits, in open country just east of Altamont, in the Santa Cruz Mountains, and on the Coast Route to Los Angeles. The Western Pacific is affected in the Feather River Canyon, in the Altamont hills, and the Niles Canyon. The

Santa Fé had trouble with its Franklin Tunnel from slide causes. The Ocean Shore is affected in a few places, notably at Mussel Rock. Mr. Dillman.

The San Francisco peninsula has a number of slides. The Sutro Baths were threatened until the slide was cured by drainage and later removal. At Lands End there is a typical slide, moving every year, which is now invading Fort Miley at its head.

One of the most evident slides, and yet one which is entirely unobserved by man, is in the Olympic Mountains. It cut an ancient lake in two, raising one part about 80 ft. to a new outlet. The place from which the material came is still very evident. The slide is now entirely covered by a heavy growth of timber. The lower lake is called Lake Sutherland, with its outlet toward the Elwha. The upper one is Lake Crescent, and its outlet is Crescent Creek.

These instances have been given in order to lead to statements of the cause and cure of slides, or, rather, the contributing cause which can be cured; and it is all very simple. The cause is water, and the cure is drainage.

The Cause is Water.—Water lubricates, and lessens friction. Water accumulates a head, and forces itself into and through otherwise impermeable material, thus extending the lubrication; but the greatest effect of water is from its pressure. It acts like millions of jack-screws, under and back of the slide, to produce motion. The film of water back of and under the slide has only to be thick enough to be continuous in order to transmit the pressure of its whole head in this manner.

We have articles on the pressure of water under dams. A slide is a dam, in all essential features, until motion begins. Then, fortunately, the continuity of the water film is broken. At the instant the continuity is sufficiently broken, motion ceases. Then, if conditions are right, the inflow of water increases the continuity of the film, flows into the cracks, and motion again begins.

A slide is frequently a number of dams, according to different planes of motion, any one of which may move. It matters not how saturated is the mass above the bottom of the slide, the analysis of bottom pressures and effects is not changed thereby. Although slides of some extent offer at first varying evidence, crumpling at the toe, upheaval in places, subsidence at the head, and lateral motion in varying degrees, they can all be traced to one phenomenon by proper analysis. There is frequently a swampy place at the head, sometimes attaining the dignity of a lake. There are usually springs at the toe, frequently also along its trace on the surface. These may develop by erosion into gulches which hide the cracks, the crumpling, and other evidences of motion.

There is no need to enlarge on the cause of slides. Every fact in evidence can be traced directly to water, principally to its pressure.

Mr. Dillman.

The Cure is Drainage.—Sometimes, the surface can be drained sufficiently to effect a cure. Surface drainage will always help; but surface drainage is often difficult, especially after motion has developed a cracked wart-like surface, as this tends to hold rainfall and guide it to the surface of motion, or several surfaces of motion.

Sub-drainage, which will kill the water pressure, is infallible. There never has been a slide that could not be cured in this way. There are cases where the expense is not warranted. There are cases where the whole slide can be sluiced away. There are also cases where the motion is so slow, or its effect so small, that the removal of the material as it comes, or not removing it at all, is the best answer. Incidentally, removal is drainage.

Subsidence at the head of the slide tends to the formation of swamps and lakes, which, in turn, supply the water to fill the cracks, to form the pressure, to produce motion, to make more subsidence, and so on in a never-ending cycle. The interruption of this cycle is most certainly accomplished by killing the head of water acting on the surface of motion. Draining the swamps and lakes will help. At Panama one enthusiast proposed concreting the whole surface of the slide to prevent the ingress of water. This might do, if there were not probably some subterranean supply of water, possibly with a great head, that would not keep out. Such construction might be an actual hindrance, instead of a help, and might serve to hold the water and increase, instead of decrease, the head.

In some cases increasing the resistance to motion has been tried, by masonry and wooden bulkheads. These have been effective where it only needed another "straw", but have generally been disastrous. Drainage by perforating the bulkhead is taught as a rudiment in retaining walls.

Far apart as they may seem, there is much similarity in slides, retaining walls, and dams. The analysis is nearly identical, gravity, friction, and hydrostatic pressure. Sub-drainage will cure the slide, is necessary to the stability of the wall, and increases the safety of the dam.

All study is for the purpose of developing or properly applying principles. The result is education. The principle of slides is: Cause, water; Cure, drainage. There are no exceptions, though, at first sight, some may appear.

Mr. Clarke.

D. D. Clarke,* M. Am. Soc. C. E. (by letter).—The writer desires to put on record his deep appreciation of the kindness of Mr. Dillman in submitting such an interesting résumé of this paper and of the whole slide situation so far as it has been developed along the Pacific Coast during recent years.

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* Portland, Ore.

Mr. Clarke.

The writer has been more or less familiar with several of the slides mentioned by Mr. Dillman, but others are not well known. It is to be hoped that some day additional information may be available regarding the Cascade Slide, and also the upheaval which occurred at Lake Crescent, Washington. The entire Profession is indebted to Mr. Dillman for his interesting statements regarding the number and cause of the slides on the Pacific Coast, and he will be quoted as an authority on this subject.

Attention is called to an interesting account of the immense slide on the Fraser River, British Columbia, by Robert Brewster Stanton, M. Am. Soc. C. E.*

* *The Engineer*, (London, England), December 14th, 1897.

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Paper No. 1416

HYDRAULIC PHENOMENA AND THE EFFECT OF SPREADING OF FLOOD WATER IN THE SAN BERNARDINO BASIN, SOUTHERN CALIFORNIA*

By A. L. Sonderegger, M. Am. Soc. C. E.

With Discussion by Messrs. Charles H. Lee, James Hyde Forbes, and A. L. Sonderegger.

Synopsis.

The San Bernardino Basin is a closed structural basin filled with detrital matter, in which Artesian conditions are created by an impervious barrier thrown across its outlet. The barrier is locally known as Bunker Hill Dike. Geologically, these conditions are the result of block faulting. (See Plate XXII.)

There are marked wet and dry seasons, and a wide range in fluctuations of seasonal as well as periodical precipitation. This is demonstrated by the residual mass curve of rainfall, constructed for a 45-year period, which curve, in its ascending and descending branches, indicates that there have been four distinct periods, of from 10 to 13 years each, of excessive and deficient rains. (Figs. 1 to 4.)

Broadly speaking, variations in precipitation call for corresponding variations in the run-off of streams tributary to the basin. (Figs. 4 and 7.)

Under conditions of natural draft, ground-water levels fluctuate in sympathy with precipitation and run-off. There is a well-defined rela-

* This paper was presented before the Southern California Association, Am. Soc. C. E., at its meeting of April 11th, 1917, and also before the meeting of the Society on October 17th, 1917.

tion between the elevation of the Artesian rim of an Artesian basin of this character, and the quantity of rising water in streams and Artesian wells, and pressure in such wells. (Figs. 5 and 6.) The theoretical head at any point in the Artesian basin is equivalent to the difference in topographical elevation between that point and the Artesian rim.

The Artesian phenomena of pressure and flow are in direct sympathy with the rainfall and run-off phenomena, as expressed in the residual mass curves. The residual mass curves of rainfall and run-off afford a safe criterion as to the position which the ground-water plane naturally should occupy, and whether artificial abstractions have materially overdrawn a basin. (Figs. 7 to 14.)

The maximum natural seepage in the river bed of the Santa Ana River occurs for a mile or two above the Artesian rim. (Table 2.) The effect of artificial seepage applied near the rim is but temporary, and the benefits increase the farther up on the débris cone the spreading is practised. The center of the spreading works on the Santa Ana cone, of late years, has been about 4 miles above the Artesian rim, and this spreading has had the effect of diminishing the seasonal drop of the water-plane above the rim by 3 or 4 ft. per year. Contrary to common opinion, it is beneficial to practise spreading, even during years of average or deficient rains, when practically no flow would escape beyond the limits of the basin, provided the application is made in the upper parts of the débris cone. (Figs. 15 to 23.)

The cost of spreading flood water varies from 5 to 20 cents per acre-ft., a maximum of 27 000 acre-ft. having been spread in one year. The average annual net conservation is estimated as in excess of 15 000 acre-ft. per year, and valued at not less than $100 000; this has been accomplished at an average annual expenditure of less than $2 000.

The San Bernardino Basin, about 70 miles east of Los Angeles, is a closed structural basin bordered on the north, east, and south by granitic and schistose mountains. To the west the valley is open, but the underground basin is closed by a subterranean barrier, known locally as the Bunker Hill Dike. Plate XXII is a general map of the basin. The dike consists of impervious clays which effectively close the basin and force practically all the underflow to the surface. The

basin is filled with alluvial deposits of gravel, sand, and clay to unknown depths, the deepest borings going to 1 150 ft. without encountering bed-rock. The deposits are porous on the débris-cones and receptive to the absorption of flood waters, or irrigation water. They are graded from coarse gravels and boulders, at the mouths of the canyons, to fine silts in the flats.

The Santa Ana River has cut a gap in the dike to a width of from 1 to 1½ miles, in which the impervious strata reach probably to within 100 ft. of the surface; and, on each side of it, the dike manifests itself as a ridge projecting above the valley floor. The effect of the dike has been to produce an Artesian basin, from which large quantities of water rise to the surface and escape, either in rising streams, or by evaporation from swamps. The maximum area of the Artesian basin in the early Nineties was 21 sq. miles. The water-shed tributary to the Artesian basin is 720 sq. miles, of which 560 sq. miles are mountainous water-sheds and 160 sq. miles are foot-hills and valley lands. The average annual water crop amounts to about 280 000 acre-ft., and the annual use is about 200 000 acre-ft.

Of the areas irrigated, there are within the basin 26 100 acres of citrus lands in the Redlands and Highland districts, and 5 000 acres of valley land planted to alfalfa and garden truck; outside, there are the Riverside Colony, with 27 700 acres, mostly in citrus fruits, and the Rialto and Fontana districts, with about 16 000 acres, a large percentage of which is also in citrus groves.

The total value of the land depending on the San Bernardino Basin for its water supply is in excess of $100 000 000.

All summer flow of the surface waters has been appropriated for many years, and during the last 20 years increasing quantities of water have been abstracted by Artesian and pumped wells. More than 50% of the water is diverted beyond the limits of the San Bernardino Basin. Notable among the concerns which export water are the Riverside Water Company, the Gage Canal Company, the Riverside-Highland Water Company, the City of Riverside, the Lytle Creek Water and Improvement Company, the Fontana Company, and others.

The underground supply of the basin is mainly derived from the seepage that occurs in the stream beds in the upper part of the débris cone between the mouth of the canyon and the rim of the Artesian basin. Except during years of extreme drought, large quantities of


Little Mountain
SAN BERNARDINO
San Bernardino
Lytle Creek
Ordecker
Meacham Well
Heap Well
McCrary Wells
Bledsoe Well
Schoolhouse
Williams Well
Gage Canal Co's Wells
Bunker Hill
Colton
Slover Mt.
Riverside Canal
Redlands
Redlands Mountains

SAN B
HIGHLAND AVE.
DEL ROSA AVE.
ARTESIAN RIM
Perris Hill
Meacham Well
Heap Well
Rising Stream
Swamps
Mc Crary Wells
Mrs. Shay
San Bernardino City
SIXTH ST.
Whitlock
Stewart
Riverside Water Co.
City Creek
City of Riverside
THIRD ST.
Frates
Bledsoe Well
Taylor
Bradley
Dutmaster
Slack
Dobbs
Swamp
CARDIFF AVE.
Davis
Shick
Huston
STILES ST.
STERLING ST.
MILL ST.
PEPPER AVE.
RIALTO AVE.
PALM AVE.
ORANGE ST.
Schoolhouse
Plunge Creek
Rising Water Channel
Williams Well
River
Santa Ana
Gage Canal Co's Wells
SAN BERNARDINO AVE.
CANAL
GAGE
COLTON AVE.
City of Riverside
Badlands Mountains

PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
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SONDEREGGER ON
HYDRAULIC PHENOMENA,
SAN BERNARDINO BASIN, CAL.

flood water escape during the flood season, beyond the Artesian rim, and beyond the Bunker Hill Dike, and are forever lost to the San Bernardino Basin.

Recently, owing to an action in the Courts filed by the City of San Bernardino against the City of Riverside and the Riverside Water Company, the hydrography of the basin and the effect of the spreading of flood waters have been made the subject of a detailed study, the writer having been one of the engineers engaged by the defendants. In order to eliminate unnecessary details, the scope of this paper will be limited to the eastern portion of the basin, which may be termed the Santa Ana cone, embracing the drainage areas and débris cones of the Santa Ana River, Mill Creek, Plunge Creek, and City Creek.

The Residual Mass Curves of Rainfall and Run-off.

There are no local rainstorms in Southern California, and the Weather Bureau records at San Bernardino may be considered typical of the distribution of rainfall in the San Bernardino Basin and watershed, relative to both time and volume. With higher altitudes there is a proportionate increase of precipitation, up to the elevation of 6 000 ft.

TABLE 1.—Monthly Precipitation at San Bernardino, California.

Month.	Depth, in inches.	Percentage of seasonal precipitation.
January	3.708	22.9
February	3.005	18.6
March	2.764	17.1
April	1.222	7.5
May	0.570	3.5
June	0.081	0.5
July	0.035	0.2
August	0.172	1.0
September	0.163	1.0
October	0.600	3.7
November	1.342	8.3
December	2.532	15.7
46-year average	16.194	100.0

The distribution of the precipitation for the year, at San Bernardino, is shown in Table 1. There is a marked difference between wet and dry seasons. The rainy months are from December to March, inclusive, during which 74% of the seasonal rainfall occurs. A tabu-

lation of seasonal rainfall at San Bernardino, from 1871 to 1916, inclusive, is shown on Figs. 1 to 4. It is difficult to grasp the general tendency of annual variation of precipitation from tables, nor are the periodical fluctuations easily discerned. In order to overcome this, residual mass curves of precipitation have been prepared for three periods: from 1870 to 1900, 1870 to 1910, and from 1870 to 1915. (Figs 1, 2, and 3.)

These curves are a diagrammatic presentation of aggregate excesses and deficiencies of precipitation; an ordinate at any point on one of them is the difference between the aggregate rainfall from the beginning of the period of record to the date considered, and the aggregate rainfall which would have fallen during the same period if the annual rate had been the observed mean.

Such a curve is partly above and partly below the zero line. It always begins near the zero line, and necessarily closes on the same, regardless of the length of the period considered. For this reason the position of the zero line is of no consequence as regards the interpretation of the curve. The curve is in descent for years of deficient precipitation, and in the ascent for years of excessive rains. If the deficiencies extend over a period of years, this is expressed in a general downward tendency, as from 1874 to 1883, and from 1893 to 1904, and *vice versa*. Thus, it will be seen that there are four distinct periods from 1874 to 1916, as indicated at the head of Figs. 1 to 4.

These mass curves, therefore, indicate the cumulative effect of precipitation, expressed in ascending and descending branches of the curves. An analysis made at the close of the season of 1899-1900, as shown on Fig. 1, would have indicated that there had been a deficiency in precipitation for the 7 preceding years, but it would not have indicated that in 1900 anything like average conditions had been reached. Compare the different positions of the point marked 1900 in Figs. 1, 2, and 3.

Fig. 4 presents the residual mass curve of run-off for the Santa Ana River for 46 years. It shows the same characteristics as the rainfall mass curve, and calls for a like interpretation. The observations from 1897 to 1916 are from measurements made by the Geological Survey; the volumes given for the years 1871 to 1897 are deduced from run-off curves. Generally speaking, the fluctuations of the run-off mass curve correspond to those of the rainfall. Deviations are due to

the fact that, for equal rains, the run-off is greater if the preceding year was one of excessive precipitation than if it was a dry year. The relation which variations in rainfall bear to those of run-off is best studied by platting the residual mass curves for both, as expressed in percentage of the mean. This is presented in Fig. 7. In this diagram the run-off mass curve represents the entire natural water crop tributary to the San Bernardino Basin, the rainfall being that for the City of San Bernardino only. It will be seen that the functions in run-off are from 50 to 100% greater than those of corresponding precipitation, both for maxima and minima.

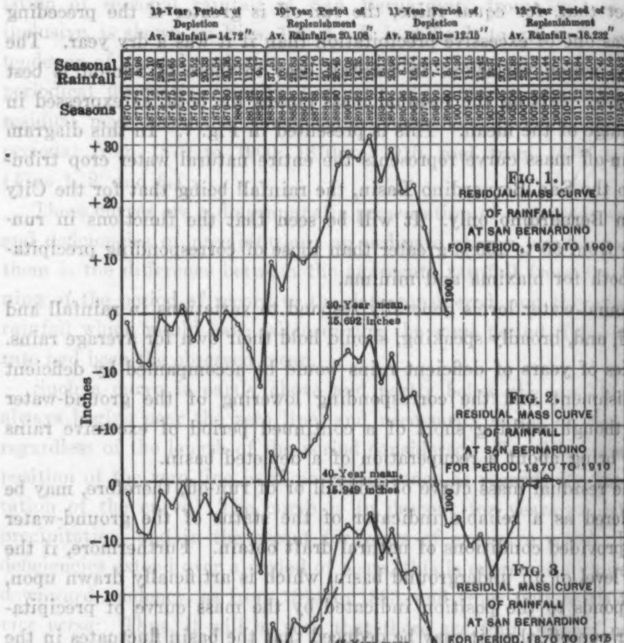
Ground-water levels generally respond to variations in rainfall and run-off, and, broadly speaking, should hold their own for average rains. A series of years of deficient rains would be accompanied by deficient replenishment and the corresponding lowering of the ground-water level, though nothing short of a continued period of excessive rains would bring about a recuperation of a depleted basin.

The residual mass curve of rainfall or of run-off, therefore, may be considered as a reliable indicator of the status of the ground-water level, provided conditions of natural draft obtain. Furthermore, if the water level of an underground basin, which is artificially drawn upon, corresponds to the position indicated by the mass curve of precipitation or run-off, then it may be deduced that the basin fluctuates in the same manner as under natural conditions, and, therefore, is not being overdrawn.

Considering the wide range in fluctuations of the seasonal precipitation in the San Bernardino Basin, ground-water levels should be subject to extensive variations from natural causes. These statements will be supported by facts given in the pages following.

The Movement of Underground Water.

If the San Bernardino underground basin had an unobstructed outlet to the west, it would produce the common phenomenon of underflow, with velocities of percolation of probably 2 miles per annum, or thereabouts. Any material contribution to such underflow at any point by means of spreading, would produce what might be called an advancing wave of underflow, and this, in due time, would be noticeable at points below by a rise in the water surface, a fact which could easily be established by observations on a series of wells below the spreading



13-Year Period of Depletion Av. Rainfall = 14.72"
10-Year Period of Replenishment Av. Rainfall = 20.108"
11-Year Period of Depletion Av. Rainfall = 12.15"
12-Year Period of Replenishment Av. Rainfall = 16.232"
Seasonal Rainfall
Seasons
Inches
FIG. 1. RESIDUAL MASS CURVE OF RAINFALL AT SAN BERNARDINO FOR PERIOD, 1870 TO 1900
30-Year mean, 15.692 inches
FIG. 2. RESIDUAL MASS CURVE OF RAINFALL AT SAN BERNARDINO FOR PERIOD, 1870 TO 1910
40-Year mean, 15.949 inches
FIG. 3. RESIDUAL MASS CURVE OF RAINFALL AT SAN BERNARDINO FOR PERIOD, 1870 TO 1915
45-Year mean, 16.007 inches
FIG. 4. SEASONAL RUN-OFF, SANTA ANA RIVER RESIDUAL MASS CURVE, 1870 TO 1915
46-Year mean, 73 900 acre-ft.
Acre-feet
1900
Total Seasonal Run-off, Santa Ana River, in Acre-feet

works. One would also be justified in crediting the underground basin with the total quantity of water spread. However, such conditions do not exist in the eastern part of the San Bernardino Basin.

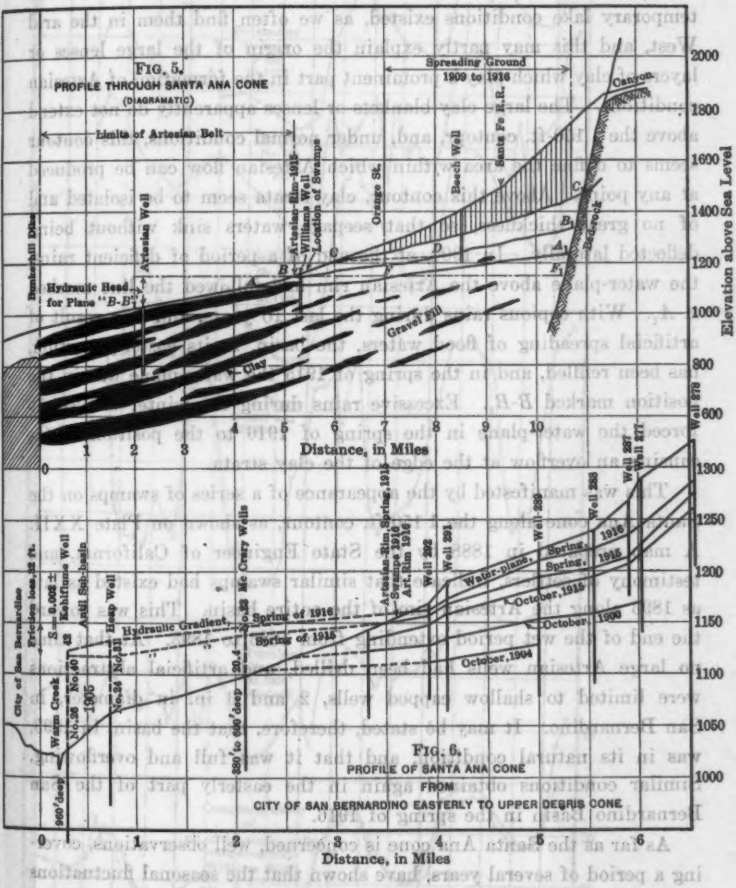


FIG. 5.
PROFILE THROUGH SANTA ANA CONE
(DIAGRAMATIC)

FIG. 6.
PROFILE OF SANTA ANA CONE
FROM
CITY OF SAN BERNARDINO EASTERLY TO UPPER DEBRIS CONE

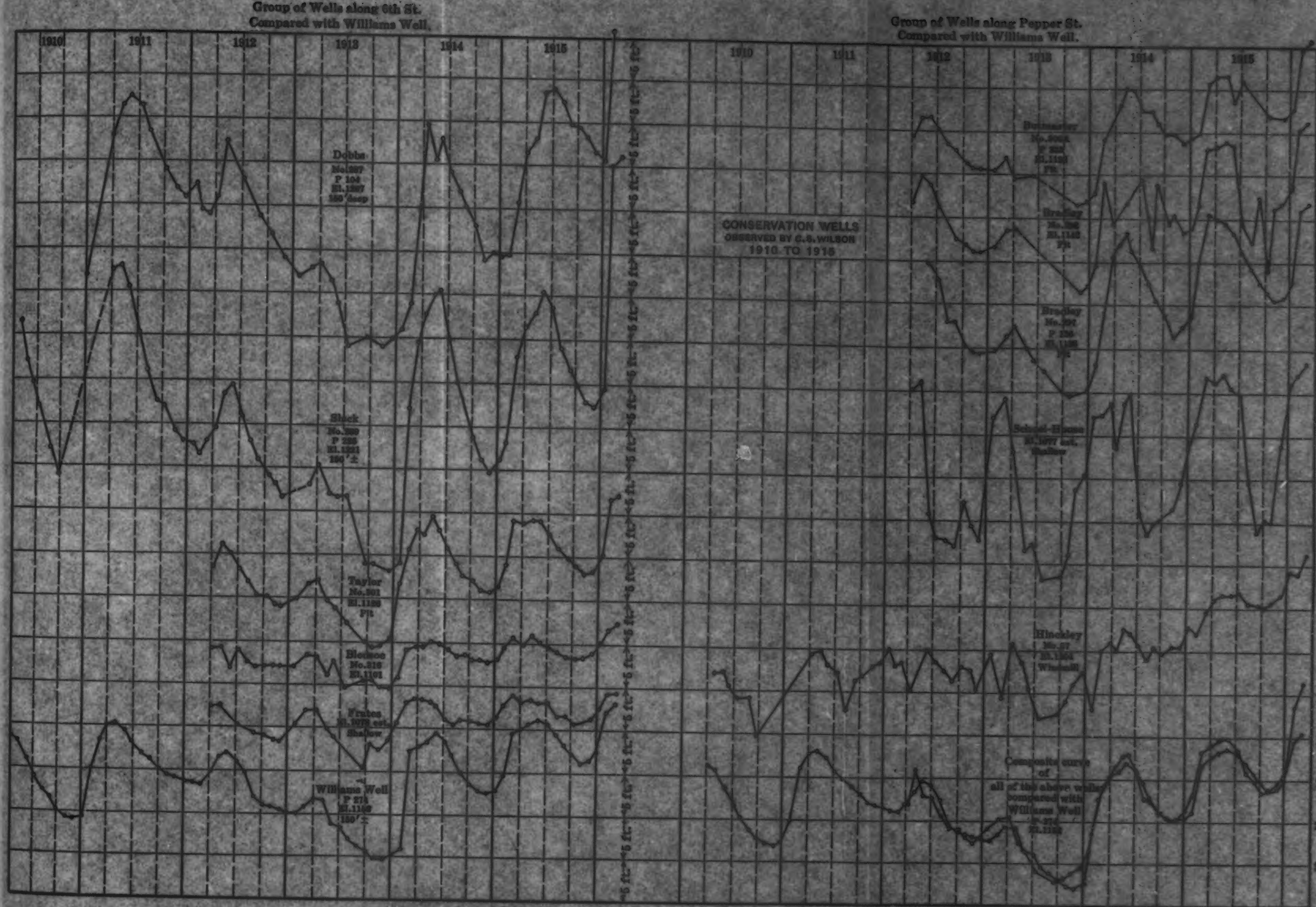
Fig. 5 is a profile of the Santa Ana cone from the Bunker Hill Dike, following approximately the course of the river. The geological features are represented diagrammatically from a large number of well-logs.

Geologically speaking, according to investigations by Professor Robert T. Hill, the San Bernardino Basin is the result of block faulting. It is a sunken valley, the fill of which consists of the alluvial deposits of flowing streams. It is probable, however, that at intervals temporary lake conditions existed, as we often find them in the arid West, and this may partly explain the origin of the large lenses or layers of clay which play a prominent part in the formation of Artesian conditions. The large clay blankets or lenses apparently do not extend above the 1 150-ft. contour, and, under normal conditions, this contour seems to define the area within which Artesian flow can be produced at any point. Above this contour, clay strata seem to be isolated and of no great thickness, so that seepage waters sink without being deflected laterally. In 1904, at the end of a period of deficient rains, the water-plane above the Artesian rim had followed the line marked $A\text{-}A_1$. With copious rains during the last 10 years, and as a result of artificial spreading of flood waters, the basin, in its easterly portion, has been refilled, and in the spring of 1915 the water-plane was in the position marked $B\text{-}B_1$. Excessive rains during the winter of 1915-16 forced the water-plane in the spring of 1916 to the position, $C\text{-}C_1$, causing an overflow at the edge of the clay strata.

This was manifested by the appearance of a series of swamps on the Santa Ana cone along the 1 150-ft. contour, as shown on Plate XXII. A map prepared in 1888 by the State Engineer of California, and testimony of settlers, indicate that similar swamps had existed as late as 1893 along the Artesian rim of the entire basin. This was toward the end of the wet period extending from 1883 to 1893. At that time no large Artesian wells had been drilled, and artificial abstractions were limited to shallow capped wells, 2 and 3 in. in diameter, in San Bernardino. It may be stated, therefore, that the basin, in 1893, was in its natural condition, and that it was full and overflowing. Similar conditions obtained again in the easterly part of the San Bernardino Basin in the spring of 1916.

As far as the Santa Ana cone is concerned, well observations, covering a period of several years, have shown that the seasonal fluctuations of the water-plane above the rim occur in the same manner as those of the surface of a reservoir, namely, as fluctuations of the water-plane as a whole, corresponding to seasonal replenishment and depletion. During the winter there is a rise culminating in April or May; and during

PLATE XXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1416.
SONDEREGGER ON
HYDRAULIC PHENOMENA,
SAN BERNARDINO BASIN, CAL.



the summer there is a drop in the water-plane terminating about October and as late as December. This is borne out by the diagrams of Plate XXIII, showing the fluctuations of a number of surface wells of less than 200 ft. depth. (For location, see Plate XXII.) It will be noted that all the wells reached their peaks, as well as the low points, simultaneously. There is a marked increase in the magnitude of the fluctuations with the higher elevations.

If there were no gravel fill above the rim, then the surface of the water-plane above the same would be horizontal, like that of a reservoir, in the manner indicated by the line, B-F-F_1, in Fig. 5, producing pressure on all points in the Artesian basin below. Owing to the leakage to the surface of large quantities of water in the Artesian belt, there must be a continuous down-stream movement of water in the entire subterranean basin, and as the gravel fill causes frictional resistance to percolation, the surface of the water-plane above the rim is not horizontal, but assumes an incline, as shown in the lines, A-A_1, B-B_1, or C-C_1.

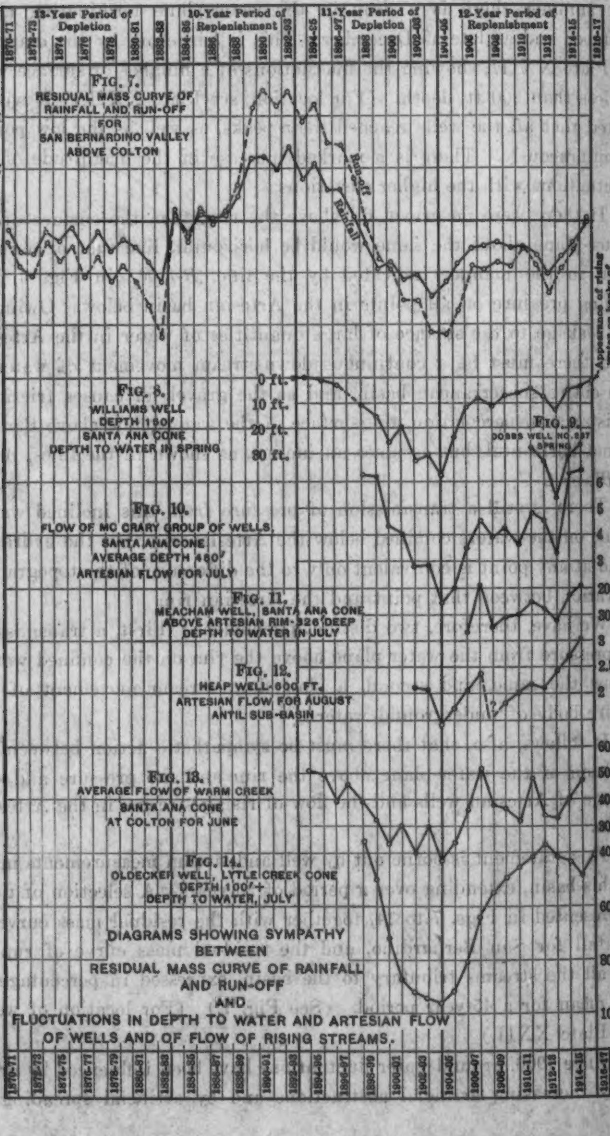
There is still a transmission of pressure from this inclined water-plane on the waters confined below the Artesian rim, but the hydraulic head at any point is equivalent only to the difference of the topographic elevation between that point and the Artesian rim.

We have, therefore, two distinct phenomena: First, a transmission of pressure from the water-plane above the rim on the confined waters below the same, and second, a slow down-stream movement of the entire body of underground water.

It follows, also, that there must be sympathetic action between the position of the water-plane above the rim and the pressure and discharge of Artesian wells and the flow of rising streams in the Artesian basin.

This statement is borne out by well and stream measurements made in this basin, extending over a period of 25 years. A selection of these is presented in Figs. 7 to 14, together with the residual mass curve of rainfall for San Bernardino, and the residual mass curve of run-off for all the streams tributary to the basin, expressed in percentage of the mean for a 45-year period. (See Fig. 7.) (For location of wells, see Plate XXII.)

Since 1900, ground-water fluctuations have been influenced by gradually increasing artificial abstractions and by artificial spreading of


13-Year Period of Depletion
10-Year Period of Replenishment
11-Year Period of Depletion
12-Year Period of Replenishment
FIG. 7.
RESIDUAL MASS CURVE OF RAINFALL AND RUN-OFF FOR SAN BERNARDINO VALLEY ABOVE COLTON
200%
100%
0
100%
200%
Run-off
Rainfall
Appearance of rising water on banks of Santa Ana River
FIG. 8.
WILLIAMS WELL
DEPTH 150′
SANTA ANA CONE
DEPTH TO WATER IN SPRING
0 ft.
10 ft.
20 ft.
30 ft.
FIG. 9.
DOBBS WELL NO. 237
SPRING
FIG. 10.
FLOW OF MC CRARY GROUP OF WELLS
SANTA ANA CONE
AVERAGE DEPTH 450′
ARTESIAN FLOW FOR JULY
Second-Feet
FIG. 11.
MEACHAM WELL, SANTA ANA CONE
ABOVE ARTESIAN RIM-326′ DEEP
DEPTH TO WATER IN JULY
Feet
FIG. 12.
HEAP WELL-600 FT.
ARTESIAN FLOW FOR AUGUST
ANTIL SUB-BASIN
Second-Feet
FIG. 13.
AVERAGE FLOW OF WARM CREEK
SANTA ANA CONE
AT COLTON FOR JUNE
Second-Feet
FIG. 14.
OLDECKER WELL, LYTLE CREEK CONE
DEPTH 100′+
DEPTH TO WATER, JULY
Feet
DIAGRAMS SHOWING SYMPATHY BETWEEN RESIDUAL MASS CURVE OF RAINFALL AND RUN-OFF AND FLUCTUATIONS IN DEPTH TO WATER AND ARTESIAN FLOW OF WELLS AND OF FLOW OF RISING STREAMS.

flood waters on the Santa Ana débris cone. (See Figs. 16 to 23, and referred to in the section following.)

Beginning with Fig. 8: This shows the highest, or spring elevation, for a period of 25 years, of the water surface in the "Williams well" of the Gage Canal Company, 6 in. in diameter, depth approximately 150 ft.; it is at an elevation of 1 152 ft., and is about 600 ft. south of the present stream bed of the Santa Ana River. The diagram may be considered as representative of the fluctuations of the water-plane near the Artesian rim and above it.

Fig. 9 gives the fluctuations for the past 5 years in the "Dobbs well"; depth, 150 ft., more or less; it is near the course of Plunge Creek, a tributary of the Santa Ana River. The elevation of the well is 1 267 ft. Its fluctuations coincide with those of the "Williams well."

Fig. 10 gives the Artesian flow of a group of five wells called the "McCrary wells", ranging in depth from 350 to 600 ft., on the banks of Warm Creek where it crosses Baseline Road; elevation, 1 093 ft. Slight deviations of fluctuations from those of the "Williams well" are explained by sympathetic action with a number of Artesian wells and pumping plants in the immediate vicinity.

Fig. 11 shows the spring elevation of the water surface of the "Meacham well", depth 326 ft., which is ¾ mile north of the edge of the Santa Ana cone, and about 1½ miles northeast of San Bernardino; surface elevation, 1 126 ft. The well was originally Artesian, the water rising to an elevation of 1 146 ft. This well is in sympathy with the deep wells of San Bernardino and the Riverside Water Company in the "Antil" sub-basin, about 1 mile east of San Bernardino; this well has been called the barometer of this sub-basin.

Fig. 12 shows the flow for August in the "Heap well" of the Riverside Water Company, situated in the "Antil" sub-basin, at an elevation of about 1 062 ft. The flow of this well is affected by that of other wells in the vicinity.

Fig. 13 shows the natural flow of Warm Creek at the Riverside Water Company's intake near Colton, for June. This discharge represents rising water, as much as 90% of which comes from the Santa Ana cone. The flow of Warm Creek is representative of the volume of natural rising water of the Santa Ana cone. (However, large volumes of water rise also in the channel of the Santa Ana River.)

The parallelism in the fluctuations of the flow of Warm Creek with those of the flow of Artesian wells, and with the water surface in surface wells above the rim, is demonstrated. The observations depicted on Figs. 8 and 13 extend over a complete cycle of dry and wet years; they indicate the return, in the spring of 1916, of conditions in the Santa Ana cone similar to those of 1893, and present a proof of the return of the water-plane to its former highest position.

In other words, the artificial draft on the Santa Ana cone has not exceeded the replenishment as it has occurred for the past 23 years, and the basin, therefore, has not been overdrawn.

The intimate relation between the volume of rainfall and run-off of the tributary streams on one side, and the fluctuations in the water-plane and Artesian flow on the other side, is easily discerned by comparison of Fig. 7 with Figs. 8 to 13.

Admitting that in 1915-16 the water-plane in the eastern portion of the San Bernardino Basin has returned to its original position of 1893—and this in the face of a very material artificial draft—then the residual mass curve of rainfall and run-off apparently does present a criterion by which the true status of the underground supply can be determined with reference to a long period of years. Without the records which are now at hand, the position of the water-plane in 1904 might have appeared exceedingly alarming, though the information which we have to-day shows that this condition was in the natural order of things, and that recuperation did follow in due time.

Fig. 14 represents the fluctuations of the water-plane in the "Oldecker well", which is 1½ miles northwest of San Bernardino, at an elevation of 1 185 ft., and outside the limits of the original Artesian belt. This well belongs to the Lytle Creek cone. Until 1911 its fluctuations, broadly speaking, corresponded to those of the residual mass curve, and to those of the other wells. In 1911-12, however, there was still a rise in the water level, which might indicate a lagging of this well. From 1912 to 1915 there was a continuous decline of the water-plane in the face of the 2 years of excessive rains of 1913 to 1915. The inference is that there was either deficient replenishment in the Lytle Creek cone after 1912, due to the diversion of flood waters at the mouth of the canyon of Lytle Creek, or that there was an excessive draft. Observations in this respect seem to point to a combination of these circumstances.

Fig. 14 tends to refute any theory which would establish the San Bernardino Basin as a unit in which there is a perfect transmission of pressure from one end of the basin to the other. It has been observed that in San Bernardino, and in the Lytle Creek cone to the west thereof, water levels in 1915-16 had not returned to their original elevations of the early Nineties, and the swamps which were characteristic of the border line of the original Artesian basin, had not returned in the Lytle Creek region.

Some observations of surface wells have been made at intervals from 1900 to 1916 by the U. S. Geological Survey. These have been used to make up Fig. 6, showing profiles of the water-plane of the Santa Ana cone for 1900, 1904, 1915, and 1916. The observations in the fall of the year present the lowest water levels for the season. There are no continuous records of pressure of any Artesian well, but for 1915 and 1916 the pressure is given for the Kehl flume well, Heap well, and McCrary wells, together with the hydraulic gradient of the water-plane of those 2 years. The friction head between the Artesian rim and the Kehl flume well for the spring of 1915 was approximately 2 ft. per thousand, and was caused by the natural leakage which appears in rising streams and swamps, as, at the time the measurements were made, but few Artesian wells were open.

Fig. 6 is in support of the general hydraulic theory which has been developed in the discussion of Fig. 5.

Another point to be brought out by Fig. 6 is that a piling up of water on the dêbris cone has occurred for the past 12 years. Storage in the dêbris cone is practicable up to the point where the basin begins to overflow over the clay blankets, as indicated by the formation of swamps.

Spreading of Flood Water.

Seepage water, if applied on the cone more or less uniformly from the mouth of the canyon to the rim, or in the upper part of the cone, would produce a simultaneous rise of the entire plane comparable to the change from the position, B-B_1 to C-C_1, Fig. 5, and the application of seepage water on the area just above the rim would produce a plane as shown by the line, C-D-B_1.

Conservation of flood water by spreading has been accomplished mainly on the cone of the Santa Ana River proper, which, with its tributary, Mill Creek, controls a water-shed of 250 sq. miles. It was

first practised by the Gage Canal Company as early as 1900, during the dry period, and a few years later by the Riverside Water Company. The method consisted of plowing the stream bed for some distance above the Artesian rim, thereby preventing a sealing up of its pores, and at the same time splitting up the stream. The effect was a change of the water-plane, similar to the one shown in Fig. 5, from the position, $A\text{-}A_1$ to $B\text{-}F\text{-}A_1$, or from $B\text{-}B_1$ to $C\text{-}D\text{-}B_1$, producing an immediate heavy efflux of rising water in the channel of the Santa Ana River along the lines, $A\text{-}B$ or $B\text{-}C$. As storage was limited to comparatively small areas situated like the area, $A\text{-}F\text{-}B$ or $B\text{-}C\text{-}D$, large portions of the water must have escaped almost as fast as they were put under ground, so that the effect of storage was only temporary. However, the Gage Canal Company enjoyed annually a material increase in the flow of the river during the early part of the summer.

The farther up the spreading is practised the more general will be the benefit on the entire Artesian basin. In 1906 the Gage Canal Company moved its spreading works into the vicinity of the Santa Fe Railway crossing, and in 1908 the Riverside Water Company began to spread water in the upper parts of the cone. Their efforts were united in 1909. In that year an organization of citizens representing these two companies, and other local water companies, was perfected and incorporated under the name of "Water Conservation Association."

Through the efforts of this Association, the Federal Government, by legislative act, set aside certain public lands in the Santa Ana cone to be used as spreading grounds. The lands were of no practical value for agricultural purposes, being covered with old river washes and partly overgrown with desert brush. The Act of Congress was under date of February 20th, 1909.*

Effective work has been carried on by the Association since 1909, there being now 2 640 acres under its control. (For location, see Plate XXII.) It has filed upon 500 sec-ft. of flood water of the Santa Ana River. A permanent concrete lodging house for laborers has been constructed on the grounds, and three concrete diversion weirs have been built, of a total capacity of 200 sec-ft. The maximum quantity of water diverted during any one day has been about 170 sec-ft. The center of the spreading works is about 4 miles above the 1 150-ft. contour.

* Statute No. 36, p. 641.

Method of Diverting.

There is no permanent dam on the Santa Ana débris cone for the diversion of storm waters, and this portion of the work has always been the most difficult. The river carries great quantities of débris during floods, with boulders many tons in weight. The construction of a permanent dam on the débris cone would sooner or later cause a material change in the course of the river, and might lead to extensive damages in the valley below. Therefore, only temporary dams are built, which are washed out with every high flood. In 1915, at the suggestion of Mr. W. E. Pedley, of Riverside, a boulder dam was constructed, from 4 to 5 ft. wide on top, with a vertical up-stream slope, and a 1 to 1 down-stream slope; height, from 3 to 6 ft.; length, about 200 ft.; and enclosed with hog-wire (mesh of No. 9 and No. 11 gauge). The dam was washed out during the same winter. A similar construction was used during the winter of 1916, special care being taken to protect the down-stream toe with large boulders, as a result of which the dam withstood the 1916 flood; but a by-pass, about 200 ft. in length, was washed out around the western abutment. The dam cost \$581 for labor, plus \$40 royalty to Mr. Pedley. The writer believes that such dams will withstand moderate floods and fulfill the purpose for which they are intended. Brush and leaves made the dam fairly tight, approximately 2 sec-ft. seeping away. However, the logical location for diversion dams is at the mouth of the canyon, where permanent structures can be maintained.

Method of Spreading.

Three methods of spreading are practised. First, from the main diversion ditch, small streams of 50 in. or less are diverted and split up into furrows, plowed by teams or made by hand. As long as no cutting sets in in the furrows, the rate of absorption is quite satisfactory, and the rivulets are not disturbed. During 1915 the maximum wetted area covered by ditches and furrows at any one time did not exceed 50 acres, and diversions varied from 120 to 170 sec-ft. The rate of absorption observed in experiments was 3.42 sec-ft. per acre of wetted area.

The second method consists of building boulder dams in the old channels of the débris cone, forming small reservoirs or ponds. They

are built from 6 to 10 ft. high, and made more or less water-tight by throwing earth against the up-stream face. Nine ponds were constructed, covering a total area of about 2 acres. The ponds, at the beginning of the season, absorb from 2 to 2.3 sec-ft. each; but their absorptive capacity decreases as the bottom becomes silted up, and at the end of the season they have to be scraped. A third method, tried as an experiment, consisted in the construction of a 5 by 5-ft. timbered pit, 40 ft. deep; cost $483. Only clear water was admitted, and the absorption did not exceed 0.7 sec-ft. constant flow.

The method of spreading by splitting up the stream into furrows is probably the most efficient, although ponds, once constructed, prove quite satisfactory, requiring but little attention. From January 1st to May 31st, 1915, one foreman, two men, and one team, were employed in spreading from 120 to 170 sec-ft. The total volume of water diverted was 26 527 acre-ft., and the total expenditure, $2 600.

Along the spreading ditches and furrows desert brush begins to die off. No water is spread during the first day or two of a storm, when the water is muddy.

It has been claimed that large quantities of water spread will evaporate. However, this is not substantiated by facts. The evaporation from still water in Southern California varies from 60 to 72 in. in depth per year, and in swamps it is approximately 96 in. per year. The percentage of evaporation during the winter, from January to May 31st, is less than one-third of the total for the year. For a wetted area of 50 acres, as observed in 1915, and assuming evaporation from spreading grounds equivalent to that from swamps, the total evaporation would not exceed 133.3 acre-ft., which, compared with the total of 26 527 acre-ft. of water spread, would be insignificant.

In order to determine the volume of net artificial absorption, it is necessary to ascertain the natural absorption which would occur in the river bed for a given stream discharge. Table 2 shows the percentage of absorption measured in the stream bed between various stations, for the season, 1914-15. (For location of stations, see Plate XXII.) The center of natural absorption is about 2 miles above the 1 150-ft. contour, that is, in the vicinity of Orange Street.

Fig. 15 shows the natural absorption curve for the Santa Ana River above the Artesian rim. It presents averages for the rainy season of 1915, and is the result of a great number of observations made under

varying conditions, and with widely differing results. Stream gaugings were made by wading, and were limited to discharges of less than 200 sec-ft. Generally speaking, the rate of absorption is much greater after heavy storms, when the stream bed has been torn up; a prolonged period of constant flow tends to silt up the pores of the channel. The percentage of absorption decreases with increased flow, because of greater velocities. On the other hand, increased depth of water and increased pressure would tend to produce greater percolation.

TABLE 2.—AVERAGE PERCENTAGE OF ABSORPTION OF FLOW OF SANTA ANA RIVER FROM THE JUNCTION WITH MILL CREEK TO THE ARTESIAN RIM NEAR PALM AVENUE.

	Absorption percentage of flow at upper station.	Absorption percentage per mile.
For a flow of 150 sec-ft. at the junction of Santa Ana River and Mill Creek the loss is as follows:		
From junction to Station 2—½ mile	4	8
" Station 2 to Station 3—0.8 mile	9.5	12
" " 3 " " 4—¾ mile	16	21.3
" " 4 " " 5—Orange St. 1½ miles	30	20
" " 5 " " 6—1 mile	30	30
" " 6 to point of rising water, ½ mile (uncertain)	5	10 (?)

Assuming now a stream discharge of 200 sec-ft., of which 150 sec-ft. are to be diverted and spread, 50 sec-ft. remaining in the stream: According to the absorption curve of Fig. 15, of the flow of 50 sec-ft., 22 sec-ft. are absorbed in the natural stream channel. The total absorption accomplished, therefore, is $150 + 22 = 172$ sec-ft. The probable natural absorption for a discharge of 200 sec-ft. would have been 108 sec-ft., leaving 64 sec-ft., or $42\frac{1}{2}\%$, as net artificial absorption. On the basis of similar computations for daily flow and diversions, Table 3 has been made up, which shows the total net artificial absorption accomplished from 1912 to 1915. The average is 48.7%, or practically 50% of the quantity of water spread.

During years of low run-off the percentage of net absorption is necessarily less, as the natural absorption is comparatively large, and but small quantities escape. On the other hand, during wet winters, in excess of 60% of the water spread presents net artificial absorption.

The benefits from spreading are twofold: First, there is the above mentioned increase in the volume of water which reaches the under-

ground basin; second, on account of the application of spread waters about 2 miles higher up on the débris cone, as compared with the location of the main natural seepage, there is an increase in the length of time for which the underflow is stored, and this applies, not only to the volume of net artificial absorption, but to the total water put under ground.

The result is a delay in the time of efflux of the water which is applied at greater distance above the rim. Some light is thrown on

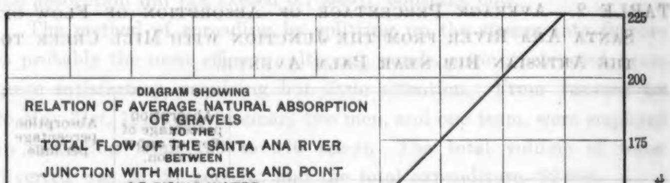


FIG. 15.

the subject by a study of the fluctuations of the "Williams well", sufficiently frequent observations of which are available, from 1898 to date. See Figs. 16 to 23. The well is 600 ft. south of the present channel of the Santa Ana River, at an elevation of 1 152 ft., and, therefore, near the Artesian rim when the basin is full. In Fig. 20 the semi-seasonal fluctuations of this well are shown, together with other data which bear on the subject.

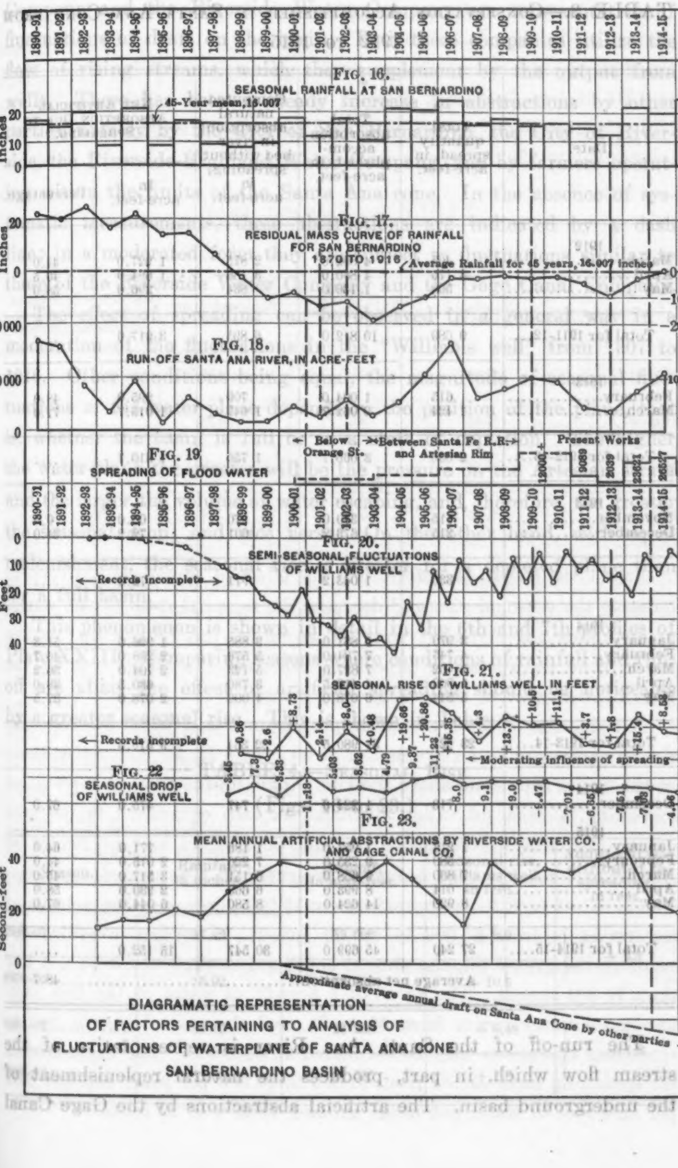


FIG. 16.
SEASONAL RAINFALL AT SAN BERNARDINO
45-Year mean, 16.007
Inches
FIG. 17.
RESIDUAL MASS CURVE OF RAINFALL FOR SAN BERNARDINO 1870 TO 1916
Average Rainfall for 45 years, 16.007 inches
FIG. 18.
RUN-OFF SANTA ANA RIVER, IN ACRE-FEET
Acre-feet
FIG. 19
SPREADING OF FLOOD WATER
Below Orange St.
Between Santa Fe R.R. and Artesian Rim
Present Works
FIG. 20.
SEMI-SEASONAL FLUCTUATIONS OF WILLIAMS WELL
Records incomplete
Feet
FIG. 21.
SEASONAL RISE OF WILLIAMS WELL, IN FEET
Records incomplete
Moderating influence of spreading
FIG. 22
SEASONAL DROP OF WILLIAMS WELL
FIG. 23.
MEAN ANNUAL ARTIFICIAL ABSTRACTIONS BY RIVERSIDE WATER CO. AND GAGE CANAL CO.
Second-feet
Approximate average annual draft on Santa Ana Cone by other parties
DIAGRAMATIC REPRESENTATION
OF FACTORS PERTAINING TO ANALYSIS OF
FLUCTUATIONS OF WATERPLANE OF SANTA ANA CONE
SAN BERNARDINO BASIN

TABLE 3.—CONSERVATION ACCOMPLISHED IN SANTA ANA CONE FROM 1912 TO 1915.

Date.	Total quantity spread, in acre-feet.	Total absorption accomplished, in acre-feet.	Computed natural absorption in river bed without spreading, in acre-feet.	NET ARTIFICIAL ABSORPTION DUE TO SPREADING.	
				In acre-feet.	Percentage.
1912					
March	4 405	4 802.0	2 875	1 927.0	44.0
April	4 176	4 880.0	3 186	1 694.0	40.5
May	508	1 130.0	834	296.0	38.0
Total for 1911-12	9 089	10 812.0	6 895	3 917.0	....
1913					
February	615	1 004.0	709	295.0	48.0
March	1 424	2 062.7	1 047	1 015.7	71.0
Total for 1912-13	2 039	3 066.7	1 756	1 310.7	....
1913					
November	115	370.0	370	00.0	0.0
December	215	673.2	601	72.2	34.0
	330	1 043.2	971	72.2	....
1914					
January	3 970	5 881.0	3 885	1 996.0	51.3
February	4 745	7 784.0	5 576	2 208.0	46.7
March	7 462	7 827.0	5 123	2 704.0	36.2
April	1 900	4 410.5	3 780	630.5	33.0
May	5 544	6 078.0	4 000	2 078.0	37.5
Total for 1913-14	23 321	31 980.5	22 364	9 616.5	....
1914					
December	718	1 222.0	747	475.0	67.0
1915					
January	1 213	1 957.0	1 186	771.0	64.0
February	4 580	9 235.0	7 220	2 015.0	44.0
March	7 800	9 668.0	6 151	3 517.0	45.0
April	4 014	8 993.0	6 663	2 330.0	58.0
May	8 920	14 624.0	8 580	6 044.0	67.0
Total for 1914-15	27 240	45 699.0	30 547	15 152.0	....
	Average net absorption				48.7

The run-off of the Santa Ana River is representative of the stream flow which, in part, produces the natural replenishment of the underground basin. The artificial abstractions by the Gage Canal

Company and the Riverside Water Company are representative of fluctuations of draft on the basin. Both these companies utilize the flow of rising streams, which they supplement by the output from wells. There has been a steady increase in abstractions by other parties, notably by the City of San Bernardino, the City of Riverside, the Riverside-Highland Water Company, and by farmers operating within the limits of the Santa Ana cone. In the absence of systematic measurements, these abstractions are indicated by a dash line; in a moderated form they are subject to fluctuations similar to those of the Riverside Water Company and the Gage Canal Company.

The effect of spreading can be observed in a general way in a moderation of the fluctuations in the "Williams well" from 1907 to 1916. Other conditions being equal, the magnitude of seasonal fluctuations of the water-plane depends on the position of the plane, that is, whether the basin is full or in a state of depletion. The higher the water-plane the greater will be the pressure on the Artesian strata and the larger the volume of water escaping, and, therefore, the greater the seasonal drop; and *vice versa*. On the other hand, with equal replenishment, the seasonal rise is greater for a depleted plane than for a full basin.

This phenomenon is shown in detail in the 6th and 7th profiles of Plate XXIII. Comparing seasons where conditions of rainfall and run-off are alike, the effect of artificial spreading should be noticeable by a greater seasonal rise. This is shown in Table 4.

TABLE 4.—SEASONAL RISE.
(Figs. 16 to 23.)

Season.	Rainfall, in inches.	Run-off, in acre-feet.	Seasonal rise. Williams well, in feet.	Additional rise attributed to spreading, in feet.
1902-03..........	17.42	59 600	+ 8.0	+ 2.5
1909-10..........	15.02	58 000	+ 10.5	
1901-02..........	11.15	26 200	+ 0.48	+ 1.3
1912-13..........	11.08	37 600	+ 1.8	

A still better criterion is represented by the seasonal drop, as shown in Fig. 22. It will be noticed that after 1907, and up to 1911, the drop became more uniform, varying from 8 to 9.5 ft. From 1911 to 1915 it has fluctuated between 6.5 and 7.5 ft. Prior to 1907 a higher water-plane generally caused a greater seasonal drop, but, subsequent to 1907, this rule does not obtain, as is shown by Table 5.

TABLE 5.—SEASONAL DROP.
(Fig. 22.)

Season.	Rainfall, in inches.	Depth to water, Williams well, in feet.	Seasonal drop, in feet.	Reduction of seasona drop attributed to spreading, in feet.
1900-01...........	17.36	20	— 11.18	....
1902-03...........	17.42	30	— 8.62	....
1910-11...........	16.40	4.5	— 7.01	1.61 to 4.17
1913-14...........	21.45	5.5	— 7.03	1.58 to 4.15

It may be stated, therefore, that the effect of spreading has been to increase the seasonal rise and decrease the seasonal drop of the water-plane above the Artesian rim. Apparently, the higher up on the débris cone the spreading process is practised the longer will be the time of storage and the greater the benefit.

A reduction of the seasonal drop of from 2 to 3 ft. at the Artesian rim is equivalent to an average of from 3 to 4 ft. for the entire basin of the Santa Ana cone above the rim. This means that, as a result of spreading, a prism of valley fill of a base area equivalent to the area of the Santa Ana cone above the rim, and of an average height of from 3 to 4 ft., has been annually preserved in a saturated condition. The area referred to is approximately 15 000 acres, and the volume of the prism is from 45 000 to 60 000 acre-ft. Allowing one-third for voids in the gravel fill, the annual beneficial storage would be not less than 15 000 acre-ft. The value of 1 acre-ft. of water in the San Bernardino country varies from \$5 to \$8, and the annual benefit due to spreading operations may be estimated at \$100 000. The expenditures for spreading have varied from \$1 000 to \$3 000 per year, and the cost per acre-foot of water put under ground, from 5 to 20 cents.

The foregoing analysis also points to the benefits to be derived from the spreading of flood waters during years of moderate and deficient rains, when the stream bed naturally absorbs all the flow, provided, however, that the spreading is done in the upper parts of the cone.

It is of interest to note that the variations in artificial abstractions from 1911 to 1915, as depicted in Fig. 23, do not seem to affect the seasonal drop, meaning that, whatever the volume of artificial abstractions may have been during those years, it was less than the quantity of water which would naturally have escaped.

The question arises whether, in view of the 3 years of excessive rains, from 1913 to 1916, the water-plane would have returned, regardless of spreading operations. Rainfall conditions during these years were very much like those of 1904 to 1907, when spreading was done, but on a small scale. The effect during these years was a rise in the Williams well, from a depth of 42.87 ft. in the fall of 1904, to 8.1 ft. in the spring of 1907, or a total of 34.77 ft. On the other hand, in the fall of 1913—that is, at the beginning of the last 3-year period of excessive rains—the water stood only at a depth of 20.57 ft. In view of this, it is believed that the water-plane in 1916 would have returned to its original elevation of the early Nineties, regardless of spreading. It was observed that large volumes of rising water, varying from 80 to 150 sec-ft., appeared in the Santa Ana River in the spring of both 1915 and 1916, indicating that in the spring of 1915 the basin was full and beginning to overflow, and that a large percentage of the spread water must have flowed out over the Artesian rim.

This fact, however, does not impair the value of spreading operations. Undoubtedly, for a number of years prior to 1915, the water-plane was maintained at a higher elevation, and parallel therewith the flow of rising streams and wells in the valley was greatly in excess of what it naturally would have been, annually shortening thereby the time during which deficiencies in Artesian flow had to be supplemented by pumping. Furthermore, if, prior to 1915, we had entered a period of dry years, the basin would have been in better condition to tide over the same.

DISCUSSION

Mr. Lee. CHARLES H. LEE,* Assoc. M. Am. Soc. C. E. (by letter).—The author has presented a remarkably clear and concise analysis of hydraulic conditions in the San Bernardino Basin, and has added a valuable contribution to scientific ground-water literature.† His long-term comparisons of residual mass curves of rainfall and run-off and well and Artesian stream-flow fluctuations, are especially illuminating. The writer was much interested to find that the author, with additional data for 4 years and well records not previously available, has reached practically the same conclusions as he himself reached in 1912 after an extended study of the situation.

The conclusions which the writer draws from the paper with regard to the local conditions in the San Bernardino Basin are:

1.—That ground-water levels and the associated phenomena of Artesian pressure, Artesian well flow, and Artesian spring and stream flow, in the San Bernardino Basin, have wide natural fluctuations corresponding to the broad variations in annual rainfall and run-off.

2.—That, thus far in its history, artificial extractions and absorption have had but a minor effect on these natural fluctuations, the predominating influence being natural rainfall and run-off variations.

3.—That, in the main, the supply of the Basin is not being overdrawn or depleted by artificial means, except in the northwest or Lytle Creek arm.

4.—That artificial additions to the ground-water supply of the Basin by water spreading on the upper portion of the Santa Ana cone have benefited that supply by bringing about a quicker return to normal conditions after the drought ending in 1904, and have provided an element of insurance against a new series of dry years.

These conclusions, with regard to local conditions, the writer believes, are all in agreement with those of the author.

A point on which the author has not dwelt, but which the writer regards as an important one from the standpoint of the future history of the Basin, is the extent of the surplus which remains undeveloped. The writer believes that, as long as swamp areas persist within the boundaries of the Artesian basin, there is a large undeveloped surplus, represented by the evaporation and transpiration losses from the moist soil and vegetation. The area of swamp land, although decreased in extent about 40% in 1904, still embraced in that year more than 10 sq. miles, and has been steadily increasing since then. The annual volume of water lost by evaporation and transpiration from the 10 sq.

* Los Angeles, Cal. Mr. Lee's discussion was presented before the Southern California Association of Members of the American Society of Civil Engineers, at its meeting of April 11th, 1917.

† Report of the Conservation Commission of California, 1912, pp. 339 to 399.

miles of swamp in 1904, represented at least 25 000 acre-ft., or nearly 70 sec-ft., during the irrigation season. This loss is capable of development by any means that will lower the plane of saturation below the ground surface beyond the capillary limit of from 6 to 8 ft. The latter may be accomplished by complete relief of Artesian pressure, or by local drainage, either with tile or shallow pumping on an extensive scale. Mr. Lee.

Passing from matters of strictly local interest, the writer recognizes two general conclusions to be drawn from the paper:

1.—The residual mass curves of rainfall and run-off are very useful in determining the natural tendencies of the water-plane and related Artesian phenomena in an underground basin. The writer would not call them safe criteria, however, without certain qualifications. First, the rainfall and run-off records should be of sufficient length to have established stable average seasonal values, otherwise the curves may be distorted too badly to interpret their meaning. Records should be at least 30 years in length, although longer periods are to be preferred. Comparison of Figs. 1 to 3 shows that slight modifications of the curve result from extending the period from 30 to 40 and 45 years. Second, the curves must be properly interpreted. It should be recognized that the zero line does not necessarily represent average conditions of ground-water level. The accumulated deficiency or excess of rainfall or run-off at the beginning of a period under investigation is usually unknown. The author has assumed it to be zero. As a matter of fact, it probably differs from zero, and the curve as a whole should be either raised or lowered with respect to the zero line. This fact does not affect the shape of the curve, however, and it is this from which the most useful information can be gained. Third, it should also be remembered that the quantity of rainfall or run-off bears no direct relation to the volume of absorption. The depleted basin might be almost fully replenished by one very heavy year, such as 1883-84, and the absorption during succeeding wet years might be small, due to the inability of the basin to receive further accretions; on the other hand, rainfall and run-off, although large during a given year, might not be excessive with respect to the volume of depletion, and a number of such years in succession might be required to fill the basin, as, for example, the period from 1904 to date. These conditions would result in a disproportionately high peak in the mass curve in 1892-93 as compared with 1915-16, and equal peaks would occur in the ground-water and Artesian phenomena. Examination of Figs. 7 to 14 indicates that the actual state of affairs was as just described. It is obvious, therefore, that no horizontal line can be drawn on a rainfall or run-off residual mass curve which will accurately represent average conditions with respect to ground-water levels. The critical ground-water stages, the rapidity of change from one stage

Mr. Lee. to another, and the general tendency during any period, however, are shown very clearly and accurately by such curves.

2.—Evidences of over-draft on ground-water supply exist in a basin, if during a period of several years of accumulating excess rainfall and run-off, such as that since 1904, there is not a corresponding recovery of any or all of the following: Ground-water level in wells outside the pressure area; Artesian pressure within the pressure area; Artesian flow from wells; and flow of springs or streams fed from ground-water, or the area of permanent swamp land or cienaga. The last of these phenomena the author has not discussed comprehensively in his paper. The writer regards it as a natural barometer of Artesian conditions, and one about which fairly accurate information can usually be obtained from residents in the locality. Evaporation from such an area is one of the natural outlets of an Artesian basin, and its area from time to time indicates the relative volumes of waste from the basin.

Taking up the paper more in detail, there are several minor points on which the writer wishes to comment.

The author states that "the hydraulic head at any point [in the surface of the Artesian basin] is equivalent to the difference of the topographic elevation between that point and the Artesian rim." The writer would qualify this by deducting pressure losses within the Artesian basin. There is continued leakage from the basin, and movement of ground-water, with accompanying hydraulic friction losses. The line of hydraulic pressure has a slope within the Artesian basin, just as in a pipe line. This is clearly shown by Fig. 6.

The author speaks of evaporation from swamps in Southern California as being 96 in., or 33% greater than from a free water surface. The writer has made extensive investigation of this subject, and believes that the author's figure is a little high. For wet swampy ground with vegetation and without an excessive accumulation of alkali salts, the writer found the losses to be about 15% greater than that from the surface of a large body of water. For wet bare soil, with the plane of saturation practically at the surface, he has found it slightly less, the quantity being more than 90% of that from a free water surface. For greater depths to the plane of saturation, the losses decrease until the limit of capillary action is reached at from 6 to 9 ft. below the surface.

The writer does not fully agree with the author in his method of determining the effect of water spreading on the Santa Ana cone. His reasons are as follows:

1.—Comparisons of the volume of water equivalent to the annual rise or lag, in feet, which the author states is due to water spreading, do not agree with the volumes of water actually spread.

The author ascribes an additional rise of the water plane, due to water spreading, of 1.3 ft. over about 15 000 acres of the Santa Ana

cone during the season of 1912-13. The actual volume spread was 3 066.7 acre-ft. (Table 3), which, assuming a porosity of one-third, would represent a depth of only 0.6 ft. instead of 1.3 ft., as the author determines from the fluctuations of the Williams' well. Again, the author concludes that the permanent result of water spreading has been to raise the average ground-water level from 3 to 4 ft. over the Santa Ana cone, above what it would have been if no spreading had occurred. The average annual volume spread for the past five seasons is 12 460 acre-ft. (Table 3), of which 50% (Table 3) would have been absorbed from the natural stream if not spread. Approximately, one-third of the latter would have been absorbed below the present spreading ground near the Artesian rim, however, and would have soon escaped (Table 2). The net annual volume absorbed as a result of water spreading, therefore, is approximately 8 000 acre-ft. Assuming a porosity of one-third, this represents a ground-water rise of 1.6 ft. over 15 000 acres, instead of from 3 to 4 ft., as estimated by the author. Mr. Lee.

2.—Conclusions based on the Williams' well alone do not necessarily apply to the whole cone. To be sure, the Williams' well is near the ground-water outlet, and its fluctuations, if unaffected by artificial draft, should be less than from places higher up on the cone. On the other hand, it is near the area in which the Gage Canal wells are situated, and is directly below and in the line of advance of water absorbed on the spreading grounds. Fully half the area of the cone is south of the spreading grounds, and not in the direction of the steepest ground-water slope. Considering the cone as a whole, the writer does not believe that the fluctuation in the Williams' well is even an approximate measure of the average fluctuation over the whole cone. The only certain method of determining the latter is from detailed ground-water contour maps based on records at numerous wells, both within and surrounding the spreading ground, and showing the ground-water elevation throughout the cone at successive dates. Such maps would afford a simple and comprehensive basis for the solution of the problem both of the quantitative benefit of spreading and also the probable area receiving the greatest benefit, if any.

The writer, however, does not wish to be understood as taking the position that the Williams' well does not show the effect of water spreading. On the contrary, he believes that this record affords evidence of the type he failed to find in 1912, namely, observations of ground-water fluctuations in the vicinity of the spreading grounds. The point on which he differs from the author is in the quantitative effect which water spreading has had on the general rise of the plane of saturation on the Santa Ana cone. The writer would be inclined to place the actual annual average ground-water rise due to water spreading at from 1 to 2 ft., instead of from 3 to 4 ft., as does the author.

Mr. Lee. This rise is greatest within the spreading grounds and directly west thereof, being comparatively small to the north and south of the spreading ground.

The important point which the author has brought out, and with which the writer agrees, is that, as a result of water spreading, an average annual volume of 12 000 acre-ft. has been absorbed on the upper Santa Ana cone, of which 50% would otherwise have flowed immediately to the ocean as flood water, and approximately 17% would have been absorbed so low on the cone that it would have rapidly escaped into the stream. The net result is the annual storage of at least 8 000 acre-ft. of water at a point on the cone from which it cannot escape for a period of 2 years or more, and is thus available as a dry-year reserve for pumping or Artesian draft from wells. The writer heartily agrees with the author that water spreading should be confined to the upper cone, and should be carried on in dry as well as in wet years. The greater the volume of flood and winter water which can be thus stored in the gravels, the more valuable will this work become.

The writer notes with interest the author's data with respect to the results obtained by the three different methods of water spreading. The relative simplicity and low cost of surface spreading would seem to indicate that the use of shafts is not desirable unless the area of the spreading ground is restricted. The quantity of water which the author states was absorbed by each shaft is less than that which the writer understands was absorbed by similar shafts in Lytle Creek Canyon. The writer was informed in 1912 by Mr. C. M. Racor, Engineer for the Fontana Development Company, that each of these shafts absorbed from 1 to 2 sec-ft.

The writer has had opportunity to investigate a number of the ground-water basins of Southern California during recent years, and has found that not all have recovered from the low-water conditions of 1904. The great coastal plain Artesian basin, for instance, remains generally in the same condition as in 1904, and, locally, the height of the water plane and Artesian pressures have diminished to even a greater extent than in that year. This is due in part to the extensive pumping developments since 1904, but more particularly to the wasteful practice of allowing Artesian wells to flow without restrictions during the winter. These wells are situated principally along the lower edge of the basin, and the water thus escaping serves no useful economic purpose.

Another basin which has not recovered and, in fact, has steadily fallen since 1904, is the Perris Valley. This is apparently due to overdraft by pumping. Much of the water thus developed is used outside the valley.

Mr. Forbes.

JAMES HYDE FORBES,* JUN. AM. SOC. C. E. (by letter).—The writer has read this paper with interest, but is somewhat disappointed at the brevity with which the author has treated some phases of a subject on which many interesting data were collected and studied. It was the writer's privilege to be associated with the author, J. B. Lippincott, M. Am. Soc. C. E., and Mr. Kingsbury Sandborn in the recent litigation between the City of San Bernardino and the City of Riverside, being engaged in the collection, study, and preparation of data for presentation in Court, 7 months of continuous residence having been spent in the vicinity studied, for 2 months of which he was at the spreading works.

As the hydraulic phenomena are entirely due to geologic conditions, the writer believes that a more detailed geologic description would add to the interest of the paper.

Geologic History of the San Bernardino Basin.—The mountains limiting the basin on the northeast are of granite of an early geologic age; in fact, they are the basement complex of this region. Lenses of secondary crystalline limestone are contained in the granite mass, but there are no evidences of late volcanic intrusions or "dikes". Southeast and northwest of the basin are other ranges of granite, and there are outcropping points of granite south of it, so that its main base or floor is composed of an impervious granite mass.

These granite ranges are of complex structure, and have been greatly faulted. The main trend of the major structure lines is northwest-southeast, in common with those of the Coast Ranges of Southern California. These structure lines were established at an early period, and later strains have adjusted themselves along the same lines. The major faults extend for many miles, following the same general trend, on each side of the basin. The great San Andreas fault, running through San Gorgonia and Cajon Passes, can be traced along the base of the mountains forming the northern limit of the basin. A minor series of structural lines lies just north and south of it. The San Jacinto fault also is a major structural feature, easily recognized from the topography, extending along the base of the San Jacinto Mountains across the San Bernardino Valley above Colton and through Lytle Canyon. About 5 miles west of the mouth of Lytle Creek, the San Jacinto fault converges into the San Andreas fault, making those two structural features of practically one zone. The steep sides of the mountains are immense fault scarps, greatly shattered and metamorphosed.

The granite masses in the San Bernardino Basin have been extensively deformed, and the deformation is expressed in a series of block faults. Massive granite is primarily an igneous magma of

* Lindsay, Cal.

Mr. Forbes. crystalline character, but it is often subjected to stresses such that the granite is broken into huge blocks which are tilted and displaced with reference to each other. In the immediate vicinity of some of the fault zones, in the instant case, the granite has been metamorphosed by intense pressure and frictional heat, and has taken the form of schists and gneiss.

As faults are joints or cracks in the rock formation along which the strata have been displaced, the result, whether the displacement is slight or great, is a break in the continuity of the strata, with a line of rupture, or a zone of fractured material, normal to the stratification. These fault zones allow a free movement of absorbed water to considerable depths, and this water may take up the heat generated by the friction of the grinding mass. The close relation of hot springs and hot wells to fault zones is generally understood, and, where fault lines are lost in alluvial valleys, they may often be traced by hot springs and wells.

The faults, marking the lines along which the main subsidence of the San Bernardino Basin block has occurred, are shown by the topography west of the valley. A fault scarp forming the south side of Cajon Canyon is the northern line; that on the south side of Lytle Creek is the southern line. The general change in elevation of the topography between is the greatest feature, indicating a subsidence. North of Cajon Canyon and south of Lytle Canyon, the elevations are 5 000 ft. and more, but between these two canyons the maximum elevation is 3 750 ft.

Previous to the deformation which resulted in the differentiation of highlands and lowlands, the topography was somewhat level and the mountains were low. This is deduced from the present topography of Perris Valley and the top of the table-land at Bear Valley. An old lake occupied the lowest and largest part of the valley. In this was deposited the fine sands and gravel which make up the earliest sedimentary formation. These sediments are sands, clays, and gravel; the sands are fine-grained and mica is abundant. The fact that the ferromagnesium minerals are not broken down through oxidation attests to deep-water deposition. In places, the clays are metamorphosed somewhat, producing shale. East of the basin this slightly consolidated stratified material has been gently folded and bent, adjusting itself to the crustal movements of the underlying granite. The period of deposition was brought to a close by crustal movements which occurred along the major fault lines previously described and the minor structure lines related to these fault zones. It was then that the San Bernardino Mountains were raised above the floor of the valley. Contemporaneous with this movement was the San Jacinto-Bunker Hill fault movement, which formed the San Jacinto Mountains and caused

the subsidence of the San Bernardino Valley between the Bunker Hill fault and the San Bernardino Mountains, forming the San Bernardino Basin. Mr Forbes.

At the end of this movement, the older alluvium was deposited by the streams which, under the new topographical conditions, had higher and more precipitous water-sheds. This older alluvium consists of extensive deposits of coarse boulders and gravel at the canyon mouths and lenses of gravels and stream-formed clay and sand throughout the valley, laid down about in the manner that the present deposition is being carried on. The red mesa lands are stream deposits, being mostly the weathered product of the eroded earliest sediments with the original abundant ferro-magnesium minerals oxidized into red iron oxide.

Further movement along the same lines followed the deposition of the older alluvium, and still continues to some extent. In old histories of the valley we find records of earthquakes felt intensely in the vicinity of the Bunker Hill and Perris Hill faults as early as 1818. The more recent displacements along the Bunker Hill-San Jacinto fault are those of December 25th, 1899, and June 20th, 1915. We find at the present time the red soil and boulder cones lifted above and tilted toward the present valley floor. This indicates the subsequent movements along the structure lines, resulting in the present height of the San Bernardino Mountains, also probably a subsequent greater subsidence of the basin trough and a rise of the Bunker Hill fault scarp.

Erosión and resulting deposition were carried on at a rapid rate, and filled the valley to its present level with the later or present alluvium, the total depth of which is unknown. Wells have been sunk to below sea level and have not encountered the granite bed-rock or even the younger shales of the earliest sedimentary period. The bulk of this alluvial fill is material laid down by streams. The streams were exceedingly irregular and intermittent in their flow, depending, as they do at present, on the sudden and capricious visitations of heavy storms. The Santa Ana River flowed west at times along the northern limit of its cone, and at such times it eroded the mesas along the base of the mountains, which were composed of coarse gravel and boulders, as is shown in the remnants still exposed. The floods picked up this coarse material and carried it farther, each successive flood moving it forward until it reached a lower portion of the valley, where, with the decrease in velocities, it was dropped. In this way the coarse gravels deposited in the Santa Ana cone are accounted for.

Stream-built slopes reflect the size and drainage area of their canyons. The short, steep canyons of the streams on the north side of the valley between the Santa Ana River and Cajon Creek have short, steep cones. Their brief torrential floods sweep the coarse débris to the mouths of the canyons, but cannot carry it far beyond, as the streams

Mr. Forbes. decrease in volume rapidly through seepage. The longer canyons, with larger drainage areas, have more gentle gradients, and their floods, being of longer duration and greater volume, advance farther into the valley. Consequently, their fans are larger and more gently inclined.

The topography is such as to have always confined the Santa Ana River within the limits of the basin, that stream having always escaped from it in or about the same position as it does now, namely, between the Box Springs Mountains and the granite hills west of Colton, Therefore, it must be concluded that this stream has been the main factor in the upbuilding of the basin. The next largest and most productive tributary drainage area is that of Lytle Creek. This creek, however, has not been as great a factor in the building up of the basin fill, because, only recently, by fault movement, has it been confined to the basin, its depositions previously having built up a considerable fan outside of the latter.

The course of each stream has always been, directly or indirectly, from the mouth of its canyon to the outlet from the valley, yet, between these points, the course has varied greatly from time to time; and this is more especially true of the larger streams. The Santa Ana River has doubtless shifted and wandered over the valley from the southern to the northern limit of the Artesian basin, flowing in widely different directions at different times and at different levels. Its old channels, in the nature of things, much broken and impaired, now lie buried beneath the hundreds of feet of later deposits. All those old channels headed in the coarse gravel of the débris cone at the mouth of the canyon, and all still lead the water ultimately, and by devious courses, to the comparatively limited outlet from the valley, as water sinking near the mouth of the canyon finds its way largely into the coarse gravels at the head of these old gravel channels, and the course that any portion of such water takes depends on the line of least resistance, which, for the most part, must be what remains of the course of the particular old channel that it happens to enter.

The position of the axis or lowest point of a valley depends on the relative size of the opposing cones. In this valley the present axis is Warm Creek, but it is reasonable to believe that, before the present course of Lytle Creek was established, the axis was farther west. The Lytle Creek cone has recently overlapped the Santa Ana cone, thus moving the axis east.

The most significant and conspicuous topographical feature of the basin is the escarpment in the valley fill of unconsolidated materials (marked as a distinct ridge where erosion has not entirely effaced it) of the Bunker Hill-San Jacinto fault. Mr. Sonderegger, on page 803, refers to the underground basin being closed by a subterranean barrier known as "Bunker Hill Dike", said to occupy a portion in the alluvium along this fault line and consisting of impervious clays which effectively

close the Basin and force practically all the underflow to the surface. From the writer's detailed study of this feature and his experience in like situations where fault movements have caused a break in the continuity of strata in alluvial materials (such as the Niles-Irvington fault across the Niles cone, in Alameda County, California), it is, to his mind, a decided misconception of the formation of the valley and of the geological agencies which led to it, to consider the Bunker Hill fault as a fold of impervious material, or "dike". Mr. Forbes.

A detailed study of material found on the fault scarp or Bunker Hill Ridge and of the penetration records of wells near the ridge convinced the writer that the materials were entirely stream deposited, for the most part sand and gravel. West of Mt. Vernon Avenue the gravels found were typical of the country rock characteristic of the Lytle Creek drainage area. Near the Santa Ana River the materials were coarser, and carried the pink gravel characteristic of the formations of the Santa Ana River drainage area.

The Bunker Hill fault has simply caused a vertical shearing and displacement of the lenses of coarse and fine material through which it passes, thus interfering with their continuity and the passage of water through them. It is often true, when fractures occur in massive bed-rock, that the overlying stratified deposits may be bent or folded to a slight degree, provided they are consolidated. It is impossible to bend or fold unconsolidated lenses of sand, gravel and clay, to form a "dike" such as is considered by Mr. Sonderegger; deformation in such cases causes an abrupt break and a shearing of the various lenses, with the result that sheared lenses of coarse gravels subsequently abut against fine material. Where the lenses of fine materials predominate in the valley fill, a deformation and shearing effectively blocks or retards the transmission of ground-water at the line of deformation. Conversely, where coarse gravel and sand lenses predominate, fault movement, taking place largely in open materials, interposes no appreciable quantity of material resistant to water movement, and percolation continues through the line of deformation at nearly the same rate as it maintains above or below that line. As has been stated previously, the Santa Ana River always left the valley approximately in the same locality, flowing at right angles to the fault trend. The result of this condition was a relatively coarse and uniform deposition in this region where the fault subsequently crossed the Santa Ana River, and the displacement, in this area, is less effective as a dam than elsewhere, because of the more open and consistent character of the deposited material. The ground-water here moved uniformly through the porous mass from bed-rock to the surface and continued to do so after faulting nearly as freely as before. Where the fault trend parallels the course of Lytle Creek, however, the ground-water levels show that it acts as a

Mr. Forbes. very effective dam, the gravel channels apparently being blocked by materials more resistant to percolation.

Many drillers' logs of wells were obtained and studied. The logs for the most part are vague, as each driller has his own idea of what should be called fine or coarse gravels and what material should be classed as clay. In some localities, due to recently made borings, the actual drillings were available. In no portion of the valley, however, was there found a continuous impervious clay stratum, as assumed by the author. The clay masses occur, not as continuous blankets, but as rather limited lenticular bodies, the records being in full accord with the recognized geologic theory of stream-deposited clays. In this case, the well logs were too scattered to correlate, except in a very general way. The sands, clays, and gravels, furthermore, are so commingled that to attempt to identify and correlate certain strata would be highly speculative. However, in the penetration records of numerous wells near the Bunker Hill fault scarp, there was recorded the presence of recent swamp materials at relatively shallow and similar depths, which might be correlated to indicate limited clay blankets in this vicinity.

It seems to the writer that Mr. Sonderegger's conception of the Artesian conditions as "phenomenal" is due to his starting with false premises, namely that the basin is closed by a continuous impervious subterranean barrier and the presence of continuous clay blankets, the border of which is termed by him the "Artesian rim". The "hydraulic phenomena" are the natural results of existing climatic and geologic conditions, climatic in that the water supply is entirely dependent on the seasonal rainfall on the contributing water-shed, and geologic in that the water is impounded underground.

Neither an impervious clay blanket nor an impervious barrier entirely closing the basin is a necessary condition for a flowing well in the San Bernardino Basin. The necessary conditions for Artesian flow are an adequate source of pressure through the body of ground-water extending to a level higher than the mouth of the well, and a retaining agent offering more resistance to the upward passage of water than the well which pierces the resistant material. (It is a well-known fact that there are Artesian wells on the sand dunes of Long Island, New York.) In the San Bernardino Basin the height of the ground-water in the apex of the Santa Ana cone, which stands at a considerable elevation above the mouths of the wells in the lower portion of the basin, exerts a pressure which acts with equal intensity throughout the cone. The frictional resistance of the porous fill retards the movements of the ground-water, causing a loss of head and establishing hydraulic grade. If it were not for the release of pressure due to the transmission of ground-water from the basin through a section of the fault line in the vicinity of the present Santa Ana channel, a uniform hydraulic grade would be maintained under the whole Santa Ana cone. Con-

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trary to this, and to Mr. Sonderegger's theory of an entirely closed basin, an hydraulic grade is established and shown by the relatively high water pressures in the deep wells in that portion of the basin locally called the Antil region (in the central part of the valley east and south of Warm Creek); a steeper grade, with its lower water pressures, is exhibited by wells of similar depth near the Santa Ana River channel.

Different water pressures exhibited by wells on the Santa Ana cone with a common source of supply are due, in some degree, to the depth of the column of resisting materials removed in drilling the well. The wells in the Antil region are from 600 to 1 000 ft. deep—the point of entrance of the water into the well casing being at the bottom, at an elevation ranging from sea level to 400 ft. above. The elevation of the water-table 2 miles east of Orange Street on the Santa Ana cone was about 1 400 ft. in April, 1915. The head or difference in elevation between a high point of the water-table and the bottom of the wells might be more than 1 000 ft. The pressure exerted by the ground-water at any point in the valley fill is that due to elevation or pressure head, with the water exerting an upward pressure and endeavoring to seek its level, as well as a horizontal pressure or velocity head. Both the velocity head and pressure head are overcome, in great degree, by the frictional resistance of the material through which the water is transmitted. A column of resisting material is removed in drilling a well, and the degree to which the pressure head becomes effective depends on the depth of this resisting column. Just why Mr. Sonderegger has eliminated the body of ground-water higher than the 1 150-ft. contour (page 811) as a source of pressure is not explained. If the frictional resistance of the porous media were a known factor, the exact source of pressure might be determined; but the writer believes that the pressure head of the higher body of ground-water east of the 1 150-ft. contour is essential to overcome the resistance and cause the water in the wells to rise above the ground surface and exhibit pressures of from 20 to 30 lb. per sq. in.

There were other wells in the same locality as the deep, high-pressure wells; those from 250 to 300 ft. in depth were designated as low-pressure wells, having Artesian flow but exhibiting pressures of only from 6 to 12 lb. per sq. in.; among the shallow wells, the water level was below the ground surface, and they are termed surface wells. This variance in water pressure and water level in the wells is attributed to the difference in elevation between the height of the source of supply and the bottom of the well casings, at which point the water, released by the removal of the resistant material, is received. It might be well to explain that in the San Bernardino Basin the general practice in drilling Artesian wells is to sink a casing to a gravel stratum and leave the bottom open, not perforating the casing at any intervening level.

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The author states (page 815) that the "friction head" between the Artesian rim and Kehl flume well was approximately 2 ft. per thousand, and was caused by the natural leakage. The writer cannot agree with Mr. Sonderegger that the loss of head due to frictional resistance of the porous fill is only 2 ft. per thousand, as he has made experiments in a limited way, in coarse sand and gravel, and has found a minimum loss of head at the rate of 1 ft. in fifty. The "Artesian rim" is that topographical contour at which the difference between the elevation of the impounded ground-water in the upper cone and the mouth of the well has sufficient head to overcome the frictional resistance of the material and cause water, released through a well, to rise to the surface. Naturally, this "rim" will fluctuate with the height of the water in the upper cone, and swamps will appear at higher levels in wet years than in dry ones. The writer believes that Mr. Sonderegger's conception of the necessity of a confining clay blanket and (page 810) that the high-water plane, *C-C*, causes an overflow at the edge of the clay strata (or Artesian rim), manifested by the appearance of swamps, is in error.

The original source of all the ground-water, whether near the surface or in deep Artesian wells, is the rainfall, and the author has presented very clearly the relation between water-level records and the cumulative rainfall record. The water-table is the surface below which the pores and crevices of earth are saturated. The elevation of this water-table varies with the vagaries of the rainfall, which governs the quantity of water available for absorption. The underground reservoir is so large that a single dry year would not show a marked decline in the water-table. Where there is an accumulation of years when the rainfall is below the average, a marked decline is seen.

Interpretation of the Water-Table Contours.—The writer has plotted the water-plane contours from actual measurements of surface wells in the valley, the latter indicating the highest point of the zone of saturation, and showing the direction of the flow, extent, and source of the ground-water. Considering the formation of the valley, the comparative size of the cones, and the plotted water contours, it was readily seen that the principal source of ground-water of the basin and of hydrostatic pressure for the deeper seated waters, is the impounded water in the Santa Ana cone, contributed by natural seepage from the Santa Ana River and controlled seepage from conservation processes. Wells on the same cone derive their supply from a common source—the saturated sands and gravels lying at the mouths of the canyons, and whatever reduces the quantity of water in that body affects, in a general way, the entire cone.

There is to be expected a decided annual variation in head in all the wells in the valley, due to the replenishment of the apexes of the cones during the winter, and a drop due to withdrawal, natural and

artificial, during the summer. The cycles of dry and wet years have an accumulative effect over longer periods. Mr. Forbes.

The topography of the water-table underlying the débris-filled valley conforms in general to that of the land surface, but the slopes are more gentle. The greatest slope of the water-table is toward the mouths of the canyons, whence comes the principal supply. However, the surface rises more rapidly, and, although the water-table in the central part of the valley is near the surface, it becomes gradually deeper in the direction of the canyons. The greatest drop in the water-table during the summer is at the mouths of the canyons. As the water maintains its hydraulic grade and is in constant motion, any depletion in the central part of the valley is supplied from the high cones, giving consideration to the retardation due to the frictional resistance of the porous media. Therefore, it is best for the economic use of the basin to draw down the water-table in order to allow the greatest replenishment, whether natural or effected by conservation processes.

The water-table has a definite gradient, and the ground-water is in a state of continual motion, as the water-contour lines indicate that there is some escape for the water at lower levels than that where it enters. Because of the pressure gradient exhibited by the Artesian water, it is known that the water in the deeper gravels also has motion. This would not be consistent with a barrier condition closing the basin, such as has been described by Mr. Sonderegger, the contour lines showing the water-table to be continuous from the basin and across the fault line at the point where the Santa Ana River crosses it to Riverside and the valley. West of Bunker Hill, on the Lytle Creek cone, there is an abrupt drop in the ground-water level from the basin toward the southwest. If a fold of impervious material existed, as suggested by Mr. Sonderegger, the basin would be like an artificial reservoir, and the ground-water might be lowered below the lip of the barrier by draft, due to pumping from the wells and depleting the stored water supply. There is no such condition in the San Bernardino Valley Basin. The lowering of the water-plane during the period from 1894 to 1904 was not an indication that the basin had been over-developed and permanently depleted, but, as clearly presented by Mr. Sonderegger through the use of residual mass diagrams of rainfall, due to the diminishing surface water supply during a cycle of dry years. The outflow from the basin continued at about the same rate as during prior years, but the supply was reduced; consequently there was a lowering of the water-table level.

The writer's contours represented the water-table position of April 5th, 1915, being that of the highest point in the replenishing period. The broad and steeply inclined underflow from the east is due to the percolation from Mill Creek, Santa Ana River, and Plunge Creek,

Mr. Forbes. greatly increased through the process of conservation carried on in the Santa Ana wash. This broad underflow conforms in shape to the débris cone. The City Creek underflow meets that from the east and adds its bulk to the main body. In fact, each of these streams has an independent cone, which broadens and flattens, coalescing at its periphery with that of the Santa Ana River, until the whole makes one body of water underlying the Santa Ana cone, with a constricted point of free escape between the Bunker Hill and Reche Canyons. The continuity of the water-table north and south of this line shows a continuous outflow from the surface to bed-rock. If there was an impervious dam entirely across the mouth of the basin, the decided drop in the water-table would be continuous. In fact, the draft in the water-table below Colton would be such as to bring about such a phenomenon. The water underneath is necessary to support the upper water in the zone of saturation.

There are streams of rising water in the San Bernardino Basin, namely Warm Creek, Towne Creek, and the lower ends of the channels of City Creek and the Santa Ana River. The ground-water emerges to the surface wherever the water level within the alluvial cone becomes higher than that of the lowest depression or level of the surface. In some confined low points the soil is soft and boggy, and swamps are formed. In the case of a continuous depression, as in the stream channels of the basin, the seepage water forms a "making" stream entering the channel above and below the water level along the sides and bottom of the channel. The writer's water-table contour lines show points at which the water-table was higher than the ground surface in April, 1915.

The water of Warm Creek is appropriated by the Riverside Water Company, which also owns numerous wells near the Warm Creek channel. When the water-table lowers, during cycles of dry years and in the latter part of each season, the flow in Warm Creek correspondingly decreases. The Riverside Water Company then allows its deep wells to flow into Warm Creek, increasing the flow to the necessary volume. The deep wells in the Antil region, owned by the City of San Bernardino, are used for municipal water supply. A number of the deep wells have a flow ranging from 100 to 125 in. of water.

The writer submits the following conclusions which he formulated after several months' study of conditions in the basin:

1.—That an impervious barrier across the entire valley above Colton is not necessary to Artesian flow, and does not exist.

2.—That there is no continuous impervious clay blanket overlying the valley. The pressure encountered in deep wells is due to the hydrostatic head, accounted for by the columns of water high in the débris cones standing at an elevation higher than the mouths of the wells.

3.—That the pressure in the different wells on the same débris cone varies according to the depth of resistant material removed and the difference in effective pressure head. Mr. Forbes.

4.—That there is a constant flow of ground-water out of this basin, but the retarding effect of a constricted point of free escape makes possible a storage reservoir.

5.—That the original source of all the ground-water is the rainfall; and the height of the water-table depends on there being an adequate supply of rainfall.

6.—That the greatest drop in the water-table occurs annually in the apexes of the débris cones, where the water is not of economic importance to the overlying lands, and that, for the economic use of the basin as a storage reservoir, this depletion should be accomplished in order to allow the greatest replenishment, natural and through conservation processes, in the flood period.

7.—That Warm Creek and other "making" streams are seepage-water streams; and that the water impounded in the valley fill, raising the elevation of the water-table to heights above the surface channels of these streams traversing the valley, increases through seepage the summer flow of these streams, and, therefore, is not waste, but of economic importance.

8.—That the lenses of gravel deposited by the Santa Ana River are in contact with the mass of coarse material at the mouth of the Santa Ana Canyon, and extend to the northern limits marked by Warm Creek and to the west beyond Warm Creek and out of the valley between the granite mountains. These lenses, acting as a porous medium for the flow of water, receive their supply through natural seepage from the waters of Mill Creek, Santa Ana River, Plunge and City Creeks, and distribute it underground to the limits of the cone.

9.—That, through the spreading of flood waters, the natural seepage may be increased on the Santa Ana cone so that there is little danger of an over-development of the basin.

10.—That the San Bernardino Valley presents the largest underground storage basin in Southern California, and the most important, in view of its natural adaptability as a reservoir and its economic development.

11.—That future development should take place north of the Santa Ana River channel and east of Warm Creek, in order to obtain the maximum supply of water.

A. L. SONDEREGGER,* M. AM. SOC. C. E. (by letter).—The statement that the evaporation from swamps in Southern California is approximately 96 in. is a deduction from experiments made in Owens Valley by Charles H. Lee,† Assoc. M. Am. Soc. C. E., relative Mr. Sonderegger.

* Los Angeles, Cal.

† *Water-Supply Paper No. 294*, pp. 60 to 63.

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to losses by absorption by plant life. He states that the loss by transpiration in freshly-cut alfalfa was 0.498 in. in depth per 24 hours, as compared with 0.3 in. of evaporation from a pan in Owens River, and of 0.38 in. from a shallow pan in the soil. Furthermore, he says:

"The results obtained by German investigators indicate the loss from sod during the growing season to be 92% greater than from water surfaces, and that from cereals to be 73% greater. Furthermore, the humidity of the air after passing over an alfalfa field is very noticeably greater than after crossing a body of water."

From these observations the writer has deduced that the loss by evaporation from swampy areas with standing water and prolific vegetation would probably be 1⅓ times as great as that from still water; however, actual experiments might lead to slightly different results.

In regard to the determination of the benefit derived from the spreading of flood water on the débris cone, more accurate results would be obtained by systematic measurements of the fluctuation in the water-plane in the upper parts of the débris cone, which are now lacking, and of pressure and discharge measurements of wells within the Artesian basin, which are only fragmentary. Much credit is to be given to the officials of the Riverside Water Company and of the Gage Canal Company, who, long ago, recognized the value of systematic well records.

The point made by Mr. Lee in regard to the net annual volume absorbed is, in part, well taken. There is an unknown portion of the quantity of artificial absorption caused by waste and seepage from the diversion ditches which tap the Santa Ana River at the mouth of the canyon—that is, at the uppermost portion of the débris cone, leading from there for a considerable distance along the margin of the cone. These ditches are not lined; during past years, owing to excessive stream flow, they have carried and wasted ever increasing quantities of water, both during the wet and the dry seasons, and it is difficult to estimate the seepage resulting from them.

The writer has recommended the co-operation of the ditch companies with the conservation association, by which these ditches may be used for diversions, for spreading purposes, enabling the application of spread-water at the very apex of the cone.

The value of spreading flood waters is rapidly becoming more generally recognized, not only for purposes of conservation, but also for flood control. With the dry rivers of Southern California, particularly in the alluvial zone, where the soil is a sandy loam mixed with disintegrated granite, and extremely easily eroded, the destructive

action of a flood occurs generally for some time after the peak has passed. It is admitted that spreading is not practicable during the very peak of a flood, and may not help to reduce such a peak, especially during floods of unusual intensity; but, after the peak has subsided, the rivers carry a moderate discharge often for weeks. Grades in the alluvial zone are generally steep—from 15 to 50 ft. per mile being very common—resulting in high velocities. The effect is that, after the peak, the water-soaked banks readily succumb to the attack by a meandering stream, and it often has happened, particularly during moderate floods, that the greatest damage occurs after the flood has subsided. To prevent such damage, spreading is as effective as it is economical, fulfilling a twofold purpose. Mr. Sonderegger.

Mr. Forbes has given an interesting description of the geology of the San Bernardino Valley. In its general features, this description is undoubtedly correct, and agrees well with the theories advanced by other geologists. However, many of the minor features and the exact chronological order of events, especially in regard to these minor features, have not been explained. Yet, many of the phenomena cannot be understood without some knowledge of the details of the geological formation and history.

The investigations which were made in San Bernardino Valley covered a period of about 2 years, Mr. Forbes participating in the work for 7 months; therefore, a great deal of information was collected and finally available, of which Mr. Forbes had no knowledge. Perhaps this is the reason that some of his deductions are erroneous. He also exhibits a tendency to draw conclusions from conditions as he found them elsewhere, notably in the Niles Canyon, in Central California. Finally, he has attributed to the author some statements which do not appear in the paper, which would indicate a somewhat superficial perusal of it.

In regard to the Bunker Hill Dike: Before attempting an explanation, it is well to review the facts, and these are as follows: A great number of wells have been drilled along this so-called Bunker Hill Dike, not only in the lower portions, where the dike forms the western barrier of the Artesian portion of the San Bernardino Basin, but also above the Artesian rim. The well drillers invariably testified that they struck a more or less impervious formation, being largely clay, or a mixture of clay and gravel, and rather compact. No water was ever struck in the dike, regardless of the depth to which the wells were drilled, and regardless of the locations of the wells, whether in the lower or in the upper part of the valley.

At the lowest point of the dike, where it is crossed by the Santa Ana River, the river has cut a number of channels or gaps into it, their total width being less than ½ mile. Underlying the different river channels the drillers found deposits of modern gravel and sand

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to a depth of about 80 ft., where they struck the impervious formation of the dike. There are several islands which separate these gaps or active river channels, and are the remnant of the clay dike projecting above the surface of the river. Their formation is identical with that which is found underlying the sand deposits in the active river channels. To all appearances they belong to an older alluvial formation which subsequently was eroded, leaving the "dike" as a remnant. Immediately above, and immediately below, the dike at this point we have the modern wash or alluvial deposit, of great depth. Above the dike the wells are Artesian; below it they do not flow. There is, as a rule, no drop in the surface water-plane across the dike, because the basins above and below the dike are full.

Going northwest from these gaps the dike appears as a surface feature, projecting above the mesa. A number of wells were drilled in this vicinity without encountering water-bearing strata. The modern Lytle Creek wash is just east of the dike, and is filled with a modern alluvium consisting of gravel and clay strata, the latter being quite prominent, and occurring in large bodies. There is some Artesian pressure in the lower part of the Lytle Creek cone, however, apparently depleted in force by an over-draft on the basin. In this vicinity the water surface above the dike, or northeast of it, stands about 70 ft. higher than below the dike, or southwest of it.

Wherever the dike material is exposed, in this vicinity, it again shows a formation different from that of Lytle Creek wash on the east side, and from some of the alluvial materials on the west side.

As we continue northwest, along the axis of the dike, the modern fill of the Lytle Creek basin changes into one largely consisting of coarse gravel and boulders, in which clay deposits become less and less prominent. The dike disappears from the surface, but is still found as a subsurface feature, and in it there is a singular absence of water-bearing strata. The drop in the water surface, across the dike from east to west, becomes more and more prominent, being 100 ft. or more; but, in this region, a new phenomenon appears in the form of at least three transverse dikes projecting from the main one, approximately at right angles thereto, and running in a northeasterly direction. These transverse dikes appear to be only spurs of ½ or 1 mile in length. They form three pockets, with as many drops in the underground plane, when measured parallel to the axis of Lytle Creek, or across these transverse dikes. Each pocket, under normal conditions, produces flowing waters, or cienagas, locally known as the Lord, Ferguson, and Rainer cienagas. The material from which these transverse dikes are formed appears to be the same as that of the main dike. It is compact and impervious, and apparently is the remnant of an older alluvium which subsequently was partly eroded, the eroded areas being filled later by the modern wash.

Mr. Walter C. Mendenhall, as geologist for the U. S. Geological Survey, investigated the San Bernardino Basin in 1903-04.* He describes the origin of the San Bernardino Basin, as follows: Mr. Sonderegger.

"A great fault runs northwest and southeast through Cajon and San Gorgonio passes and along the base of the San Bernardino Mountains. In the movement along this fracture a portion of the earth's crust north of the present valley, was uplifted and now forms the San Bernardino Mountains, while the valley itself represents a great depressed area south of the fracture.

"Another crustal movement, whose beginning at least may well have been contemporaneous with the first, both being very late geologically, resulting in the uplifting of a ridge—the formation of an irregular arched wrinkle—extending from the San Jacinto Mountains northwestward along the line of the Badlands, which separate San Timoteo Canyon from San Jacinto Valley. The rocks which were folded into this arch are soft shales and sandstones and gravelly alluvium, like that deposited by the rivers now in San Bernardino Valley. This fold can be traced on the surface as the Bunker Hill dike to a point nearly two miles somewhat south of west of San Bernardino. It probably extends even farther in the direction of Lytle Creek Canyon as an underground feature, buried beneath the modern wash, but there is no surface indication of its presence there.

"This clay and gravel ridge has been the most effectual of subsurface dams, against which the modern stream wash has accumulated, and behind which the waters percolating seaward through this wash have been stored, the excess rising in springs and flowing over the dam, to sink again in the sands and gravels below."

Mr. Mendenhall, therefore, defines the Bunker Hill Dike as a fold, consisting of alluvial deposits, of earlier date than the modern wash.

Of late years an additional and very extensive investigation has been made by Professor Robert T. Hill, of the U. S. Geological Survey, particularly with reference to the fault lines of Southern California. This work has not been published as yet. However, Professor Hill testified in regard to the Bunker Hill Dike before the Courts, the following being a copy of part of the reporter's transcript; pages 6967 *et seq.*

"A.—I think that the mouth of Lytle Creek, as the canyon ends at the mountain bed, shows at least three different stages of deposition: First, the old alluvium, which we have already discussed, occupying the high bench to the west side of the mouth of the river; a canyon was subsequently cut into that alluvium during a later epoch, following uplifts, and radiating out from that canyon was a second formation, which we have known as the Rialto fan; it is a talus fan, fan-shaped in distribution, and with an apex up toward the mouth of the canyon.

"Q.—Is that the fan that exists on the surface of the earth to-day?

"A.—It is; and Lytle Creek has radiated as far in various directions as I am indicating with my pencil. It has flowed back and forth across

* *Water-Supply Paper No. 142*, "Hydrography of San Bernardino Valley, California", p. 30.

Mr. Sonderegger. this fan in the way I have demonstrated with my pencil. The third stage in the history of the drainage at the mouth of Lytle Creek, below its mouth, is the abandonment of that old fan, and the adoption of its recent and present course to the east of the line which we speak of as the 'Bunker Hill Ridge.' And this has below it considerable modern alluvium, making the trend of the three alluvial formations which I have understood you to ask about.

"Q.—You differentiate the alluvial deposit to the north and east of what is known as the Bunker Hill Dike from the alluvial deposit to the south and west?

"A.—I do.

"Q.—That deposit to the north and east of the Bunker Hill Dike being the more recent?

"A.—It is.

"Q.—And that to the southwest being the more ancient?

"A.—It is.

"Q.—What is the Bunker Hill Dike?

"A.—Excuse me for laughing; that is one of the most difficult questions which has been asked to answer. The Bunker Hill Dike is the name used for a feature which has expression in the surface topography, and which, as I understand from the testimony of Mr. Finkle, which I have read, and discussions with the Hydrologist, beneath the ground forms a barrier in the San Bernardino Basin to the southwest, in a way. It seems to be a feature which is properly acknowledged and admitted, but which it is very difficult to define. We do know that, when you look to the northwest from the hill seen at the east of the mouth of Reche Canyon, that you can see the topographic expression of this feature along a line to the northwestward in a general direction toward the mouth of Lytle Creek. This is indicated by hills, small hills, on either side of the Colton Road, and also by a change in the direction of float to the west side of the escarpment which Lytle Creek follows from its mouth southeasterly. That is the course in which the Lytle Creek of to-day is flowing. This feature is in line with two very conspicuous structural features to the south, one of which is the continuation of the San Andreas fault line trend to the northwest. The other is the fold which continued that line to the eastward known as the Bunker Hill Dike. Both of these features are trending to the southwestward. Again, we have coming southwesterly in that general direction the hypothetical extension of the great Lytle fault which I have described to-day. * * *

"A.—Now this feature spoken of as the Bunker Hill Dike, north of the street which I know as the Foothill Boulevard, is practically an eastward facing escarpment. The creek follows the valley at the foot of this east facing escarpment. An escarpment which faces toward the mountains in this manner is geologically known as the inface, and I would describe the Bunker Hill Ridge, as I prefer to call it, instead of a dike, north of the Boulevard before mentioned, as the inface escarpment toward the San Bernardino Plateau at the northeast. Now, there isn't visible in the structure of the material of this portion of the dike between the mouth of Lytle Creek and the Foothill Boulevard any evidence, either of the dike as geologists know it,

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of an intrusion of a different material, of one material into another material, along the fault line, an intruded wedge or block; nor is there any other structural evidence, so far as I can make out, though I found in one place slight deformations which would give evidence of an actual fault along that line in that portion of its course; though I believe in general the feature lies along the projection of the San Jacinto and Lytle faults in a way, so far as they can be harmonized. * * *

"Q.—I now return to my previous question: What was the character of the material you found in the surface evidence of the Bunker Hill Dike?

"A.—The material which I found upon the dike at the house which I mentioned just northerly in the course of the dike, of the tablet of the old Mission site—I don't know who lives in that house, a red-roofed house—the material there is all Lytle Creek drift and gravel and wash, typical Lytle Creek material, consisting of schists largely and also carrying a specimen of rock which I have only found in my examination of this vicinity as coming from Lytle Creek, and I would like to have that stone in evidence.

"Q.—Have you it with you?

"A.—I have.

"Q.—Let's see it.

"The Court.—That will be marked Exhibit O-4.

"Mr. Irving.—You have now produced a rock, which you have described as what, Mr. Hill?

"A.—It is a very peculiar porphyritic andesite."

Later, Professor Hill described the dike again as belonging to the middle formation of the three which he mentioned, and which he describes as the Rialto fan; that it was deposited after the San Jacinto fault line was formed; that the fan made and covered over the evidence of the extension of the fault line; that the chief evidence of that fault line beneath that plain to-day is the hot springs, otherwise than the evidence which the hydrologist may have (and which he was not discussing).

The theory advanced by Professor Hill affords a satisfactory explanation of all the phenomena, and, therefore, we may define the Bunker Hill Dike as the remnant of an old alluvial deposit, more compact than the modern wash, and practically impervious; and, incidentally, overlying a fault line which can be traced at least in its lower reaches.

There are many buried dikes in Southern California; Mr. Mendenhall, in other papers, has referred to them as the remnants of an older topography.

The writer has gone into the details of this geological discussion on account of the importance of these underground features with regard to the hydrology of Southern California. Artesian basins form a conspicuous feature of the local ground-water supply, and they are numerous in the San Fernando, San Gabriel, Pomona, and Santa Ana Valleys. Generally the Artesian flow is the result of the occurrence

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of buried dikes; but there are also numerous instances where Artesian flow and rising water are produced merely by a contraction of the valleys in the shape of narrows, where the rivers have cut through mountain or hill formations which have been thrown up across their courses. The most conspicuous of these narrows is the one in the San Gabriel River at Whittier, which is more than 2 miles in width, and in which the modern wash extends to a depth of 1 000 ft. or more without encountering bed-rock or any older alluvial deposits. Above these narrows there is an Artesian basin of considerable extent, and Artesian or rising streams are produced which furnish 30 sec-ft. or more during the dry season.

The Courts have taken cognizance of the existence of underground dikes, treating them in the same manner as the division lines of surface drainage areas, and enjoining the exportation of water from one side of such dikes to the other, where the existence of an impervious stratum was proved, either by geological evidence, or a drop in the water-plane. One of the leading cases in this regard is that of Burr *vs.* McClay Rancho, in the San Fernando Valley.

As far as the phenomenon of Artesian flow in the San Bernardino Basin is concerned, it is of no consequence whether the dike is a solid barrier, or whether the interruption of percolation is due to a vertical shearing or dislocation of alluvial deposits. The writer has not made the statement attributed to him by Mr. Forbes, that a solid clay barrier is a necessity in producing Artesian conditions. He has stated the occurrence of the dike as a fact, and the existence of Artesian flow in connection with that dike.

Another point disputed by Mr. Forbes is the occurrence of large clay blankets in the San Bernardino Basin. The existence of extensive clay bodies or sheets has been known for many years to the engineers of the Gage Canal Company, as attested by the exhibits which they produced in the Courts, and these were based on the results obtained in drilling innumerable wells. Mr. Mendenhall, who investigated this phase in detail, writes:*

"Within the limits of the present and the original (Artesian) basin, the characterizing feature is the presence of extensive sheets of clay."

The writer has never claimed that clay blankets are a necessity for the production of Artesian flow; but, in the San Bernardino Basin they do exist, and give rise to the phenomena which are peculiar to this basin.

Mr. Forbes, in the course of his discussion, makes the following rather startling statement:

* *Water-Supply Paper No. 142*, p. 38. He gives a well section on p. 40, and a cross-section of the San Bernardino Valley on p. 32.

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"If it were not for the release of pressure due to the transmission of ground-water from the basin through a section of the fault line in the vicinity of the present Santa Ana channel, a uniform hydraulic grade would be maintained under the whole Santa Ana cone."

The phenomenon of a uniform hydraulic grade presupposes the existence of uniform percolation from one end of the basin to the other, and this in turn would necessitate, first, a uniform pressure head, uniform grade, uniform frictional resistance to percolation, and the absence of any leakage to the surface. Only one of these conditions is fulfilled, and that is the uniform elevation of the pressure head (provided we limit the discussion to the Santa Ana cone). There are certain localities, in the San Bernardino Basin in general, and the Santa Ana cone in particular, where the underflow is apparently more prolific than in others, producing better wells and stronger rising streams. The writer refers to the "Antil Basin", east of the City of San Bernardino, and the territory or basin in which the Gage Canal Company's wells are situated, which is 2 or 3 miles southeast of the Antil territory. These two basins of exceptionally prolific flow are separated by a zone where wells produce less Artesian flow, and where the Artesian or rising streams experience less rapid increase. This is undoubtedly the result of the existence of a zone of denser deposits. It is the very absence of uniformity in the formation, and the occurrence of such zones of more prolific flow, which have given rise to endless litigation, because everybody wants to locate his wells in these zones.

The only escape from the basin over the dike, which has been definitely established, is, as Mr. Forbes indicates, that which occurs in the vicinity of the present Santa Ana channel, where, overlying the dike, we find gravel and sand strata of a total width of about 1 000 ft., and a depth of about 80 ft. Assuming 33% voids, and velocities of percolation of about 2 miles per annum, the seepage over the dike would be less than 10 sec-ft., a quantity which is small in comparison with the total quantity of water which leaks vertically to the surface above the dike, appears in the rising streams, and finds its way out of the valley as a surface stream. These rising waters amount to more than 100 sec-ft. of constant flow. A study of the pressure in various wells indicates that, in the vicinity of the dike, they are not affected by this gap, provided they reach a depth of several hundred feet.

Mr. Forbes furthermore states that he cannot agree with the writer that the "friction head" between the Artesian rim and Kehl flume well is only approximately 2 ft. per 1 000, and is caused by the natural leakage. He states that he has made experiments in a limited way, in coarse sand and gravel, and has found a minimum loss of head at the rate of 1 ft. in 50. The statement by the writer is not a theo-

Mr. Sonderegger. retical one, or based on experiments, but is the result of a very simple mathematical operation based on pressure observations on a line of wells, as indicated in Fig. 6. This friction loss of approximately 2 ft per 1 000, not only occurs between the Artesian rim and the Kehl flume well, but also between the McCrary and Heap wells. This is one of the facts which the writer considered as of general interest. It would seem useless to discuss papers and establish theories if the elementary facts are disregarded. Mr. Forbes' experiment probably was made under conditions of free percolation in the absence of Artesian pressure.

However, assuming Mr. Forbes' contention to be correct, that a friction head of 1 ft. in 50 is required to produce percolation, where would such an assumption lead? The distance from the mouth of the canyon of the Santa Ana to the point where the river crosses the dike is about 11.5 miles, or 60 250 ft., this being the shortest distance along which percolation can occur. With a friction loss at the rate of 1 ft. in 50, the total head required to produce such percolation would be 1 205 ft. The actual difference in elevation of the surface of the ground between the two points is 925 ft., the mouth of the canyon being at Elevation 1870, and the Bunker Hill Dike crossing at Elevation 945. In order, then, to produce a friction head of 1 205 ft., as required by Mr. Forbes' theory, it would be necessary to place the fountain head 280 ft. above the mouth of the canyon, or at Elevation 2150.

The supposed frictional resistance of 1 ft. in 50, alone, therefore, would require more than all the head available, due to the difference in elevation of the extreme ends of the basin.

However, there is still another factor to be taken care of: namely, the head required to produce the Artesian pressure, which pressure, in some wells, has been as high as 100 ft., which would bring the supposed fountain head to an elevation of not less than 2 250. This elevation is found about 3 miles above the mouth of the canyon, in a narrow gorge; also on the slopes of the peaks near the mouth of the canyon.

The following is a brief bibliography on the subject:

Residual Mass Curve of Rainfall:

W. E. Spear. "Study of the Water Supply Sources of Long Island." Report of Comm. on Additional Water Supply of New York City, Appendix 7, 1903, pp. 753 and 817.

Report of Conservation Commission of California, 1912. Charles H. Lee. "Subterranean Storage of Floodwater by Artificial Methods in San Bernardino Valley."

Mr. Sonderegger.

Evaporation:

Water-Supply Paper No. 294. "An Intense Study of the Water Resources of a Part of Owens Valley". Charles H. Lee.

Agricultural Bulletin No. 177. "Evaporation Losses in Irrigation." S. Fortier.

The San Bernardino Basin:

Water Supply Papers Nos. 59 and 60. J. B. Lippincott. "Development and Application of Water near San Bernardino, Colton, and Riverside."

Water Supply Paper No. 942. Walter C. Mendenhall. "The Hydrography of San Bernardino Valley, California."

"Irrigation in Southern California." William Hamilton Hall, State Engineer of California. 1888.

U. S. Department of Agriculture, *Bulletin No. 9.* Office of Experiment Stations. E. W. Hilgard, "Report on Irrigation Investigations for 1901."

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Paper No. 1417

THE HELL GATE ARCH BRIDGE AND APPROACHES OF THE NEW YORK CONNECTING RAILROAD OVER THE EAST RIVER IN NEW YORK CITY*

By O. H. Ammann, M. Am. Soc. C. E.

With Discussion by Messrs. W. H. Breithaupt, Leon S. Moisseiff, Samuel T. Wagner, Charles Evan Fowler, Henry H. Quimby, Henry B. Seaman, Gustav Lindenthal, Clement E. Chase, and O. H. Ammann.

Synopsis.

The principal object of this paper is to present an account of the design and construction of the Hell Gate Arch Bridge over the East River in New York, a structure of imposing magnitude, with unusual features and details of unprecedented size which mark a decided advance in bridge engineering.

The Hell Gate Bridge, which forms part of the New York Connecting Railroad, is the greatest arch bridge built to date, having a span of 995 ft. 1⅜ in. between centers of bearings and 1 017 ft. between faces of abutments, and a total height of 305 ft. above mean high water. It carries four railroad tracks on a heavy ballasted floor. Its principal features, besides its great span and capacity, are the exceptional size and weight of its individual members and riveted connections, the use of special high-carbon steel, the unusual method of erection, and the monumental towers forming the abutments, one of which rests on a deep and difficult pneumatic caisson foundation.

* Presented at the meeting of November 21st, 1917.

The introductory part of this paper is devoted to the object, history, and a brief general description of the New York Connecting Railroad.

The Hell Gate Bridge is the result of many years of careful and laborious studies involving the design of different types of bridges. These various preliminary designs are briefly described and critically discussed, and the conditions which led to the final adoption of the spandrel-braced arch type are set forth.

The description of the Hell Gate Bridge, as built, is confined to the more important features and details, with particular reference to the reasons and considerations which governed their design, and with a further view to bring out their merits and illustrate the progress made in bridge design in recent years. Typical details are shown in the accompanying illustrations and therefore need no particular description.

With due consideration of the fact that the strength and durability of a bridge are gauged by the strength and efficiency of its details, most careful attention has been given to their design, and a number of important improvements over common practice can be recorded. Among the details deserving special mention are the compact closed section of the main arch rib, the extraordinary rigid bracing, the efficient latticing of compression members, the full splicing of compression joints, and the provision of heavy braking girders to relieve the floor-beams from stress due to the braking and traction forces.

It has been the aim of the administration of the Railroad Company to produce a first-class railroad in every respect. This principle has been observed throughout the design and construction of all its structures. The design has been governed by rules and specifications especially prepared by the consulting engineer. These differ in many respects from common specifications, and contain many original features. Extracts from these rules and specifications are given in this paper, with critical remarks on the selection of the material, assumptions of the various loads and forces, and of the permissible unit stresses.

Among the original features of these specifications may be mentioned a new formula for impact which, in combination with apparently high permissible unit stresses, is applicable to the design of bridges of any length of span or any capacity, and secures in each case well-proportioned structures.

The paper further contains a detailed account of the essential features of the fabrication and erection of the Hell Gate Bridge which called for the highest grade of workmanship, required special tools and machinery of exceptional size, and involved unusual erection problems. This account is intended to illustrate the advance in the fabrication and erection of large bridges of the riveted type. Especial mention may be made of the complete assembling of the arch trusses at the shop, and the extensive utilization of parts of the permanent structure for erection purposes.

The approaches to the Hell Gate Bridge, which comprise the East River Bridge Division of the New York Connecting Railroad, consist of several heavy truss bridges, a double-leaf bascule bridge, about 10 800 lin. ft. of plate-girder viaducts with concrete piers, and about 3 200 lin. ft. of embankment between high retaining walls and concrete arches. Space does not permit of their detailed description, but their unusual and original features, and the reasons and conditions which governed their design, are related.

The calculation of stresses in the arch trusses, as given in Appendix A, although presenting no new theory, will be found of value to those who may have to solve similar problems. It illustrates the application of the modern theory of elasticity in the shortest and most convenient form.

The writer is under obligation, for permission to present this paper and for valuable information, to Gustav Lindenthal, M. Am. Soc. C. E., the Consulting and Chief Engineer, to whose plans and direction the successful completion of the Hell Gate Bridge and Approaches is due.

For convenience of reference and discussion, the paper is written under the following principal headings:

1.—Object and History of the New York Connecting Railroad;
2.—General Description of East River Bridge Division;
3.—Development of Design of Hell Gate Bridge;
4.—General Arrangement, Proportions and Cost of Arch Bridge;
5.—Masonry Towers and Foundations;
6.—Details of Design, and Weight of Steel Superstructure of Hell Gate Bridge;
7.—Camber and Deformation of Arch Trusses;
8.—Material;
9.—Loads and Unit Stresses;

10.—Workmanship and Fabrication;

11.—Erection of Hell Gate Bridge;

12.—Approaches;

13.—Track Floor Construction;

14.—Engineering Organization.

Appendix A.—Calculation of Dead-Load, Live-Load, and Temperature Stresses in Arch Trusses of Hell Gate Bridge;

Appendix B.—Financing, and Franchise of the New York Connecting Railroad Company.

1.—Object and History of the New York Connecting Railroad.

Object.—The New York Connecting Railroad is a part of the comprehensive plan of the Pennsylvania Railroad, inaugurated 15 years ago by the late A. J. Cassatt, then President of that Company, and which had for its principal objects the extension of the line from New Jersey into New York City and direct rail connection, for passengers and freight, with Long Island and the New England States.

The parts of this general project which established entrance into New York City, and connection with the Long Island Railroad, comprising the tunnels under the North River, Manhattan Island, and the East River, the great passenger terminal in Manhattan, a large terminal yard, called "Sunnyside Yard", in Long Island City, a freight terminal yard, at Greenville, N. J., and various other improvements have been fully outlined and described in detail.* (See also the map, Fig. 1).

The New York Connecting Railroad establishes a physical connection between the Pennsylvania Railroad and the New York, New Haven, and Hartford Railroad Systems, and thus provides continuous rail communication, through New York City, between Canada, the New England States, and the South and West.

The connection with the New Haven Line is made at Port Morris in The Bronx. From there, the Connecting Railroad crosses the East River, *via* Wards and Randalls Islands, and joins the Pennsylvania Railroad at Sunnyside Yard in Long Island City, whence the trains pass through the East River Tunnels to the Pennsylvania Station at 34th Street, in Manhattan. This connection *via* Sunnyside Yard,

* *Transactions*, Am. Soc. C. E., Volumes LXVIII and LXIX (1910).

however, is intended for passenger trains only, inasmuch as the tunnels under the North and East Rivers and the Manhattan Station can now accommodate only passenger traffic.

The freight connection will be made by another branch of the New York Connecting Railroad, which after crossing the East River at Hell Gate, passes through Long Island City, Woodside, Winfield Junction, and Middle Village, to Fresh Pond Junction, near the Brooklyn Borough line, where it connects with the Bay Ridge Branch of the Long Island Railroad. From Bay Ridge, the freight cars are transported on car-floats across New York Bay to the Greenville Yards of the Pennsylvania Railroad in New Jersey. Eventually, a tunnel may be driven under New York Bay from Bay Ridge to Greenville, which would provide also for freight by continuous-rail connection from New England to the South and West through New York City.

Besides this through freight traffic, the New York Connecting Railroad will accommodate the bulk of the freight transportation to and from the large manufacturing districts which have been developed during recent years in Brooklyn and Queens. For this purpose a number of freight yards have been established along the Bay Ridge Branch of the Long Island Railroad.

Although not doing away entirely with car-float and ferry service, the New York Connecting Railroad will greatly relieve the inner waters of New York Harbor of this traffic, particularly of the obstructive car-floats which now transfer freight from the Pennsylvania Railroad terminals on the Jersey shore of the Hudson River, up the East River to the New Haven Railroad freight terminals at Port Morris and Oak Point, in The Bronx.

Traffic.—The early plans contemplated a double-track line. Later, it was realized that the probable future development of an extensive passenger and freight traffic between the New Haven and the Pennsylvania Systems would soon require four tracks. Estimates also proved that it would be much more costly to add two tracks in the future than to build a four-track line in the first place. The entire portion from Port Morris to the point in Long Island City where the two passenger tracks branch off toward Sunnyside Yard was built therefore for four tracks, the remainder being for two tracks only.

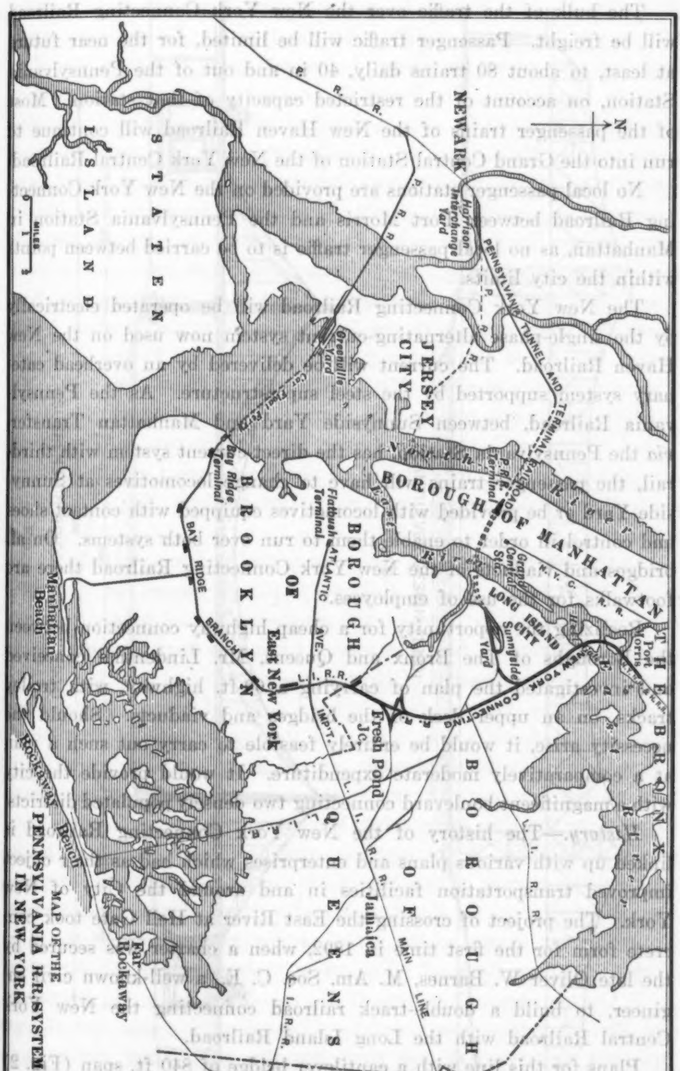
MAP OF THE
PENNSYLVANIA R.R. SYSTEM
IN NEW YORK
NEWARK
JERSEY CITY
STATEN ISLAND
BOROUGH OF MANHATTAN
BOROUGH OF BROOKLYN
BOROUGH OF QUEENS
THE BRONX
LONG ISLAND CITY
Harrison Interchange Yard
Greenville Yard
Bay Ridge Terminal
Flatbush Terminal
Sunnyside Yard
Port Morris
Fresh Pond Jc.
East New York
Jamaica
Manhattan Beach
Rockaway Beach
Far Rockaway
PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD
NEW YORK CONNECTING R.R.
BAY RIDGE BRANCH
MAIN LINE
MILES

Fig. 1.

The bulk of the traffic over the New York Connecting Railroad will be freight. Passenger traffic will be limited, for the near future at least, to about 80 trains daily, 40 in and out of the Pennsylvania Station, on account of the restricted capacity of that station. Most of the passenger trains of the New Haven Railroad will continue to run into the Grand Central Station of the New York Central Railroad.

No local passenger stations are provided on the New York Connecting Railroad between Port Morris and the Pennsylvania Station in Manhattan, as no local passenger traffic is to be carried between points within the city limits.

The New York Connecting Railroad will be operated electrically by the single-phase alternating-current system now used on the New Haven Railroad. The current will be delivered by an overhead catenary system supported by the steel superstructure. As the Pennsylvania Railroad, between Sunnyside Yard and Manhattan Transfer, *via* the Pennsylvania Station, has the direct-current system with third-rail, the passenger trains will have to change locomotives at Sunnyside Yard or be provided with locomotives equipped with contact shoes and control in order to enable them to run over both systems. On all bridges and viaducts of the New York Connecting Railroad there are footwalks for the use of employees.

Realizing the opportunity for a cheap highway connection between the Boroughs of The Bronx and Queens, Mr. Lindenthal conceived and investigated the plan of carrying a 60-ft. highway, with trolley tracks, on an upper deck of the bridges and viaducts. Should the necessity arise, it would be entirely feasible to carry out such a plan at a comparatively moderate expenditure. It would provide the city with a magnificent boulevard connecting two densely populated districts.

History.—The history of the New York Connecting Railroad is linked up with various plans and enterprises which had as their object improved transportation facilities in and around the City of New York. The project of crossing the East River at Hell Gate took concrete form for the first time in 1892, when a charter was secured by the late Oliver W. Barnes, M. Am. Soc. C. E., a well-known civil engineer, to build a double-track railroad connecting the New York Central Railroad with the Long Island Railroad.

Plans for this line with a cantilever bridge of 840 ft. span (Fig. 2) over the East River at Hell Gate were worked out by the late Alfred

P. Boller, M. Am. Soc. C. E., as Consulting Engineer, and construction was to be started in 1900. At this time the Pennsylvania Railroad had mapped out its scheme for extending its line from New Jersey into Manhattan. The North River was to be crossed on a suspension bridge of 3 100 ft. span, which had been designed and promoted previously by Mr. Lindenthal. The bridge would have accommodated ten railroad tracks, rapid transit and highway traffic, and would have required the participation of all the other railroads terminating in New Jersey. The bridge was to be connected by an approach descending with a 2% grade to a tunnel under 32d Street, commencing near Tenth Avenue and extending across Manhattan and under the East River to a connection with the Long Island Railroad System. This tunnel was called the Steinway Tunnel, after the President of the Tunnel Company of which Mr. Barnes was the Chief Engineer. Mr. Lindenthal mapped out, to Mr. Barnes, a connection of the Long Island System with the New Haven Railroad System in The Bronx on substantially the present line of the New York Connecting Railroad, thus establishing a continuous line through New York City territory.

As the other railroads in New Jersey declined to cooperate in the North River bridge project, this was finally abandoned by the Pennsylvania Railroad in favor of two single-track tunnels, the previous objections to tunnels for train service having by that time been overcome by the introduction of electric operation.

In 1900 the Pennsylvania Railroad acquired control of the Long Island Railroad, and a connection with this road across the East River and with the New England roads, via the New York Connecting Railroad on the line originally mapped out, became the natural necessary adjuncts to the plan of entering Manhattan. As a result, the Pennsylvania Railroad, in conjunction with the New Haven Railroad, acquired the charter of the New York Connecting Railroad. Samuel Rea, M. Am. Soc. C. E., then Vice-President of the Pennsylvania Railroad, took charge of the entire project as President of the New York Connecting Railroad. The entire work was under his direction; A. J. County, Assoc. Am. Soc. C. E., was Assistant to the President; and Mr. Lindenthal was appointed Consulting Engineer and Bridge Architect to work out new plans for the East River Bridge and Approaches. The surveys and location in detail, as well as the first

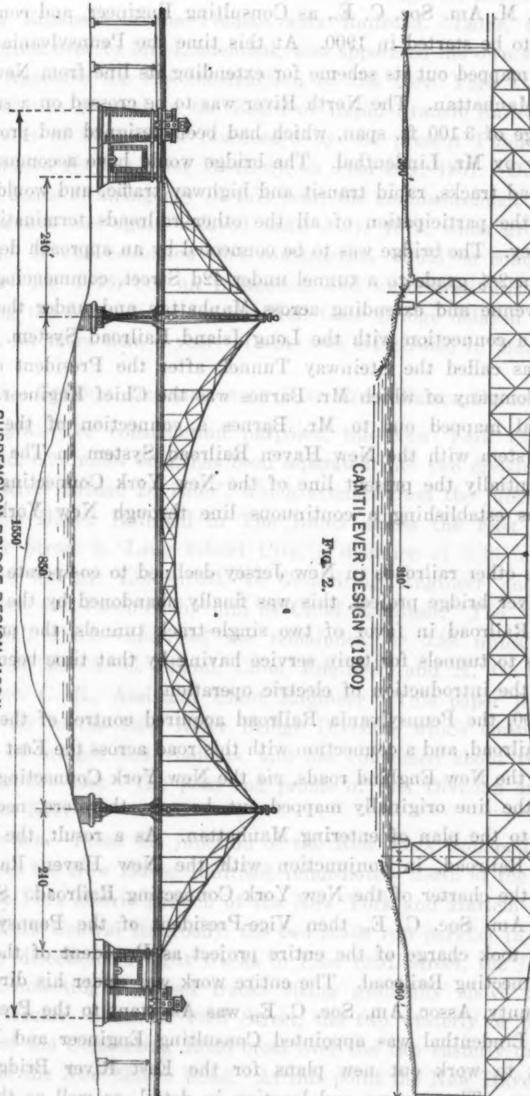

FIG. 2. CANTILEVER DESIGN (1900)

FIG. 3. SUSPENSION BRIDGE DESIGN (1904)

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borings for the foundations, were made by Joseph N. Crawford, Consulting Engineer of the Pennsylvania Railroad. Later, this work was turned over to Mr. Lindenthal, who appointed his own staff.

A franchise for the construction of the New York Connecting Railroad was granted by the Board of Rapid Transit Railroad Commissioners of New York (succeeded by the present Public Service Commission for the First District), in February, 1907. On account of the financial stringency in this year, the commencement of actual construction was again delayed until July, 1912, when ground was broken. It was estimated that construction would be completed by January 1st, 1916, but an injunction and unforeseen difficulties in building the foundations for the Hell Gate Bridge delayed the completion for more than a year. Traffic was established in March, 1917.

2.—General Description of East River Bridge Division.

Length.—For construction purposes, the New York Connecting Railroad, 8.96 miles long, has been separated into two divisions. The "East River Bridge Division", which extends from the junction with the New Haven Railroad in The Bronx across the East River to Stemler Street in Long Island City, a distance of 3.38 miles, was designed by Mr. Lindenthal, as Consulting Engineer and Bridge Architect, and executed by him as Chief Engineer. The "Southern Division", which comprises the remainder, or 5.58 miles, was in charge of Mr. A. C. Shand, Chief Engineer, and H. C. Booz, M. Am. Soc. C. E., Assistant Chief Engineer. This paper is devoted exclusively to the East River Bridge Division, which consists principally of bridges and viaducts, and has consumed about two-thirds of the entire cost. The plan and profile of this Division are shown on Plate XXIV.

Location.—From the junction of the New York Connecting Railroad with the New Haven Railroad, immediately north of the crossing over the Port Morris Branch of the New York and Harlem Railroad (New York Central Railroad), the two roads run parallel in a southwesterly direction on eight tracks to East 133d Street, the four New York Connecting Railroad tracks rising gradually above the New Haven tracks. South of 133d Street, the two westerly or passenger tracks of the Connecting Road cross over the two easterly or freight tracks of the New Haven Road. At this point the New Haven tracks

turn on a sharp curve toward the northwest, ending in the Harlem River Station and Yard, opposite 125th Street, Manhattan.

The New York Connecting Road continues in the southeasterly direction, crosses Bronx Kill on a double-leaf bascule bridge, and then skirts the east shore of Randalls Island on a viaduct 1 965 ft. long. Another arm of the East River, called "Little Hell Gate", between Randalls and Wards Islands, is bridged over by four skew deck truss spans, of an aggregate length of 1 154 ft.

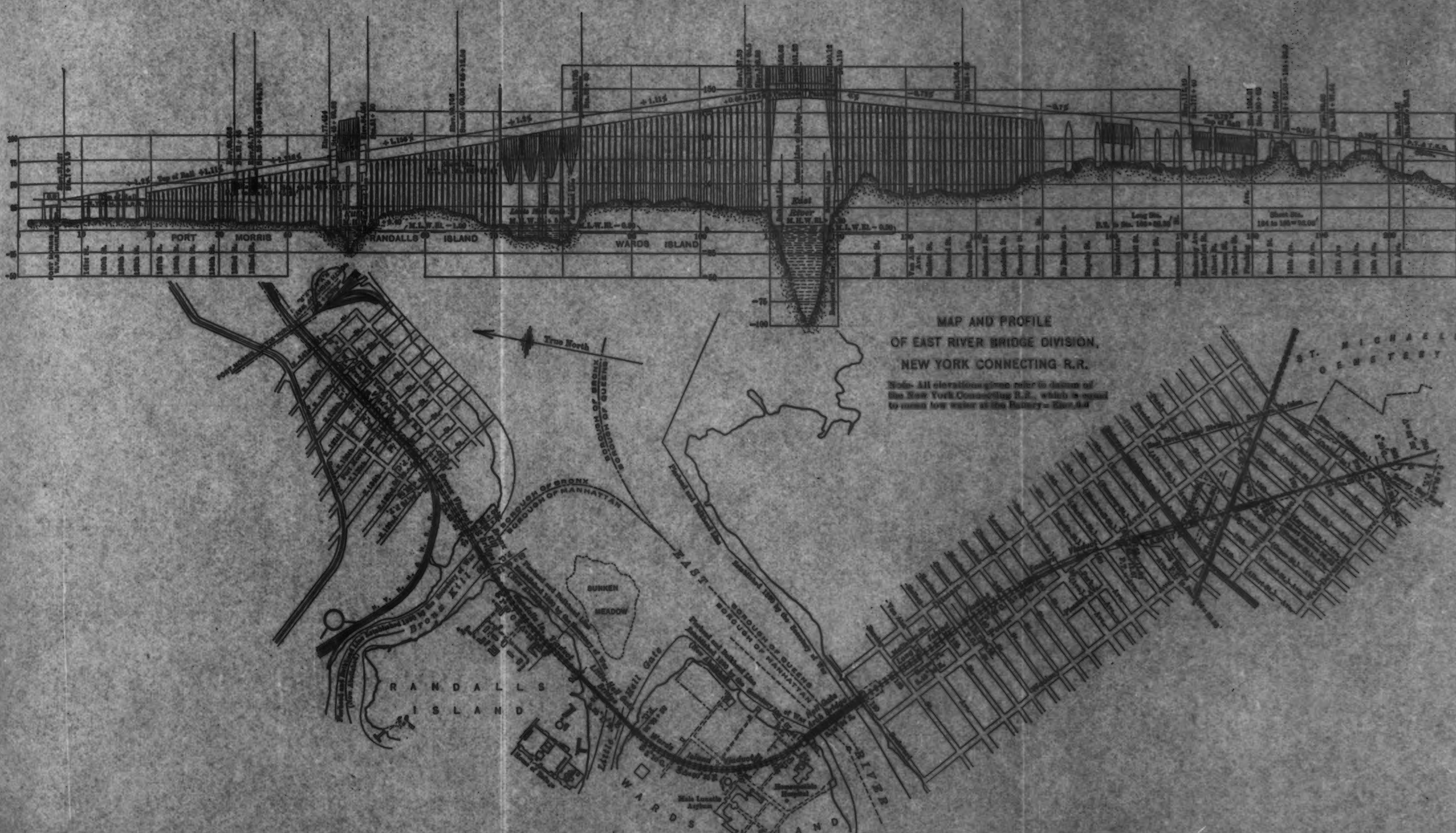
On Wards Island the line turns to a southeasterly direction with a sweeping curve on a viaduct from 95 to 130 ft. high and 2 654 ft. long. The main channel of the East River is crossed at "Hell Gate" by an arch bridge having a single span of 1 017 ft. between abutment towers, a total height of 305 ft. and a clear head-room of 135 ft. above mean high water.

On the Long Island side of the East River, the road continues in a southeasterly direction, descending partly on a viaduct, and partly on an embankment, from a height of 110 ft. to 30 ft. above ground at Stemler Street, Long Island City, where the Southern Division begins.

The total length of the main line is 8.96 miles, of which 3.73 miles have four tracks and the remainder two tracks. A study of the map and of the river conditions as outlined herein will show at once that this route, crossing the East River at its narrowest point, was the natural one to follow. Any other route would have been much more expensive. A tunnel under the East River at this point would have been a very expensive and hazardous undertaking, and would have deprived the passengers of the picturesque and more comfortable ride over the elevated structure.

Curves and Grades.—The maximum curvature is 4° for a short distance at 133d Street, The Bronx. On Wards Island, the curvature is 3° 10′ for a length of 2 545 ft. All other curves are less than 1 degree. North of Hell Gate Bridge, or against south-bound traffic, the grade is nearly uniform, 1.2% (maximum 1.218%), compensated on curves. This grade was governed by the elevation of the tracks over the East River, where the clear height of 135 ft. above mean high water was prescribed by the War Department; this is the same clearance as that for the other East River bridges. South of Hell Gate Bridge, the tracks descend on a uniform grade of 0.72%, the ruling grade for north-bound traffic.

PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1417.
AMMANN ON
HELL GATE BRIDGE.



Quantities and Cost.—The New York Connecting Railroad will be one of the most expensive railroad lines ever built. Its total cost, inclusive of right of way, will be about $27 000 000. Of this, the East River Bridge Division consumed approximately $18 500 000, or $5 500 000 per mile of four-track line. This division required about 500 000 cu. yd. of granite and concrete masonry and 90 000 tons of steelwork.

3.—DEVELOPMENT OF DESIGN OF HELL GATE BRIDGE.

Introductory.—A great work of art evolves from an idea in the mind of its creator. It is brought on paper or into a more contemplative form and then changed and remodeled. Not until the plans have passed through changes and corrections, and have been submitted to an almost endless series of finishing touches, does the great work attain its perfection.

A great bridge in a great city, although primarily utilitarian in its purpose, should nevertheless be a work of art to which Science lends its aid. An elaborate stress sheet, worked out on a purely economic and scientific basis, does not make a great bridge. It is only with a broad sense for beauty and harmony, coupled with wide experience in the scientific and technical field, that a monumental bridge can be created. Fortunately, the Hell Gate Bridge was evolved under such conditions, and therefore may well be said to be one of the finest creations of engineering art of great size which this century has produced.

As mentioned heretofore, under "History of the New York Connecting Railroad", the first design for the Hell Gate Bridge was made in 1900 by the late Alfred P. Boller. It was a cantilever design with a central span of 840 ft., supported on braced steel towers. (Fig. 2.) The bridge was designed for two tracks only, for a light live load, approximately equivalent to Cooper's E-40, and for open tie flooring.

From 1904, when Mr. Lindenthal was appointed by the Pennsylvania Railroad to work out new plans on more modern lines, until 1912, when actual construction was started under his direction, the design of the Hell Gate Bridge received almost continuous and thorough study, involving the working out of complete designs of several types of bridges and a number of modifications of the type finally adopted.

River Conditions.—The East River is an estuary or tidal stream forming the eastern entrance to New York Harbor from the Atlantic Ocean by way of Long Island Sound. That part immediately east of its confluence with the Harlem River, between Wards Island and the Long Island shore, is the so-called "Hell Gate," which name is due, evidently, to the great dangers and trying conditions to which navigation was formerly exposed at this spot. On account of the presence of many protruding rocks and reefs, the sharp bend of the channel just below Hell Gate, and the rapid tidal currents, which even now attain velocities of 7 miles per hour, collisions and disasters in this locality were of frequent occurrence. These conditions have been greatly improved by the removal of the most dangerous reefs, some of which required the blasting away of enormous quantities of rock. The most famous operation was the removal of "Flood Rock", in 1885, when nearly 300 000 cu. yd. of rock were broken up in one blast.

The river at the Hell Gate Bridge is 850 ft. wide between shore lines and 700 ft. between bulkhead lines, as established by the War Department. Both shores fall rapidly to a greatest depth of 105 ft. below mean high water. The mean tide is 5.7 ft. At present, the channel has a minimum depth of 26 ft., but, eventually, it is to be dredged to a depth of 35 or 40 ft. so as to permit the safe passage of deep-draft vessels. The river traffic is quite considerable, consisting to a great extent of car-floats and tows which are difficult to control.

Types to be Considered.—The river conditions, as outlined, and the great height of the tracks above the water, prohibited physically and economically the construction of any permanent or even temporary support in the river channel, and called for a single river span of at least 850 ft., and of a type that could be erected without the use of falsework in the river. The only types which can be taken into consideration under such conditions are the cantilever or its relative, the continuous truss, the stiffened suspension bridge, and the arch (hingeless, two, or three-hinged).

Judging from prevailing tendencies, most engineers undoubtedly would have considered the cantilever type as best suited to the existing conditions, and it is not surprising, therefore, that the first design was of that type.

The suspension type for railroad bridges is considered by many engineers as unsuitable for spans of less than 2 000 ft., and, perhaps,

very few would have looked to the arch as a suitable type in this case, because it is usually associated with steep rocky shores which afford natural solid abutments and cheap anchorages for erection back-stays.

The span length, required clearance, character of soil, and other local conditions at Hell Gate are such that, in a broad sense, there is little if any difference in cost between the several types mentioned. Whatever differences in cost may be found by comparative designs are largely due to the individual judgment of the designer in the selection of the truss system, material, permissible unit stresses, foundations, and architectural features.

A real economy of the suspension type over the others comes in with spans greater than 850 ft.; an appreciable economy of the arch over the cantilever and suspension types would have been realized with more favorable configuration and character of ground, particularly if the required clearance had permitted the same span length for the arch as for the other types. There was the more reason, therefore, for selecting the type for the Hell Gate Bridge on broader than mere economic principles.

Mr. Lindenthal conceived the bridge as a monumental portal for the steamers which enter New York Harbor from Long Island Sound. He also realized that this bridge, forming a conspicuous object which can be seen from both shores of the river and from almost every elevated point of the city, and will be observed daily by thousands of passengers, should be an impressive structure. The arch, flanked by massive masonry towers, was most favorably adapted to that purpose.

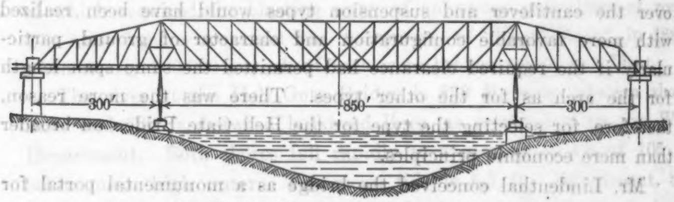
Comparative Designs
of Cantilever, Continuous, and Suspension Types.

In 1904 Mr. Lindenthal made comparative designs of the three types comprising the stiffened suspension type with eye-bar chains (Fig. 3), the three-span continuous truss (Fig. 4), and the three-span cantilever (Fig. 5), all with a central span of 850 ft. and a total length varying from 1450 to 1550 ft. The designs were made both for two and four tracks, and open tie flooring. The live load assumed was the Pennsylvania Railroad standard loading of 1904, which is approximately equivalent to Cooper's E-50.

Nickel steel was assumed for the trusses in the four-track designs only, and ordinary carbon steel for the floor system, bracing, and towers

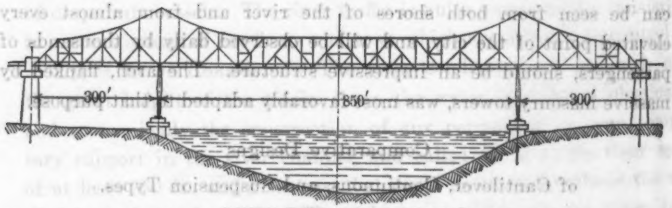
in the four-track designs and throughout for the two-track designs. The estimated weights of steelwork varied from 7 000 to 8 500 tons for double-track and from 11 200 to 14 200 tons for four tracks (if carbon steel had been assumed these weights would have been 14 000 and 17 000 tons, respectively), being least for the suspension bridge and greatest for the cantilever.

The saving in steelwork in the suspension design was partly offset by the greater cost of the anchorage piers, but, under assumed favorable soil conditions, the estimates showed a saving in cost in favor of the latter design. Under the more unfavorable soil conditions actually found on the Wards Island side, the total cost would be more nearly alike for the different designs.



CONTINUOUS TRUSS DESIGN (1904)

Fig. 4.



CANTILEVER DESIGN (1904)

Fig. 5.

Suspension Design.—The system of trusses adopted for the suspension design (Fig. 3) is that of an inverted three-hinged spandrel-braced arch suspended from hinged towers. The upper chord or chain of eye-bars follows very nearly the equilibrium polygon for dead load. The web members and lower chord form, with the main chain, the stiffening trusses.

Owing to the hinge at the center, the system is statically determinate and immune to settlements of the foundations. A similar system,

but without the center hinge, was used by Mr. Lindenthal in his design for the bridge over the St. Lawrence River, at Quebec, made in 1898, and for the Manhattan Bridge over the East River in New York City. This system has been fully described and discussed by him,* and, therefore, will not be further explained here.

The system used in Mr. Lindenthal's design for the Quebec Bridge, made in 1910,† although similar in form, differs from the previously mentioned system in that its two chords or chains form intersecting catenaries equi-distant from the line of equilibrium for dead load. This system is applicable to very long spans only, where the tension in the chains from dead load cannot be reversed by the live load.

In the design for the Hell Gate Bridge a hinge was provided at the center, as it was not known whether solid rock foundations could be obtained at reasonable depth. Wherever the piers rest on unyielding foundations, it is preferable to omit the hinge. In comparison with the bridge types shown in Figs. 4 and 5, the suspension type, with its graceful outlines, possesses unquestionably the advantage of more pleasing and monumental appearance. The anchorage masonry and the main towers give opportunity for architectural treatment. The appearance would be improved by a slight increase in the length of the end span.

In point of rigidity, the cantilever, in general, is superior to the suspension bridge. However, with the system of stiffening trusses selected for this design and the great sag of the chains (one-sixth of the span length), the deflections of the suspension bridge are reduced to about one and one-half times those of the cantilever. This greater deflection, however, is no serious disadvantage in a bridge of such size and capacity, in which the dead load is more than twice as much as the maximum assumed live load and more than four times the live load under average traffic conditions. Moreover, in a suspension bridge, the deflections from live load are free from that jerkiness which is the disagreeable characteristic of deflection in the cantilever bridge system.

Although unusual, the erection of the eye-bar chain suspension bridge, without falsework in the center span, is entirely feasible, and does not present more serious difficulties than that of a cantilever.

* *Transactions*, Am. Soc. C. E., Vol. LV (December, 1905).

† *Engineering News*, November 23d, 1911.

It was particularly for its monumental character, together with its apparently smaller cost, that the stiffened suspension bridge was recommended by the designer for adoption in preference to the other types, and it would undoubtedly have been executed had construction been started at that time and had not certain conditions developed later which led to the study and final adoption of the arch type.

Continuous Truss.—In point of rigidity, the continuous truss (Fig. 4) is superior to the cantilever and suspension type. As regards appearance, however, this design is the least satisfactory. Both for economy and appearance the design could be improved by depressing the top chord so as to make the height at the middle of the center span about two-thirds of that over the main piers, and by using a single instead of the antiquated double web system. In any case, however, the appearance would be that of a utilitarian structure.

The common objection to a bridge of this type is that the trusses are statically indeterminate and may be affected seriously by settlements of the foundations. Where solid foundations can be reached at reasonable depth, this objection, however, is not valid, and this type may be found to be cheaper and more suitable than other types if esthetic considerations are of no importance. The erection of a continuous truss bridge, being similar to that of a cantilever, does not present unusual difficulties.

Cantilever Design.—The cantilever design (Fig. 5) is similar to that shown in Fig. 2, except that slender hinged towers take the place of the unsightly braced towers of the latter design. Although superior in appearance to many existing cantilever bridges, both designs produce the effect of utilitarian structures inherent to cantilever bridges. There is no opportunity for monumental towers or abutments at the ends, because the absence of a large horizontal thrust or pull does not justify a large mass of masonry at those points, as in the case of an arch or a suspension bridge.

The hinged towers in the design (Fig. 5) are a distinct improvement over the braced towers, in that they eliminate dangerous secondary stresses which are caused in the case of the latter. A further decided improvement as regards appearance, rigidity, and permanence, and, therefore, fully justifying the slightly greater expense, would be the substitution of solid masonry piers for the steel towers below the bottom chord of the trusses. Such piers have the further advantage

that the longitudinal forces from braking and traction can be transmitted through them to the foundations on the shortest way and without increasing appreciably the size of the piers. In the design (Fig. 5) the anchor arms have the proper length, whereas in the design (Fig. 2), they are too long, both as regards economy and as the ends are subject to reversal of reaction from live load which causes objectionable "hammering."

Conditions Which Led to Investigation of Arch Type.

In 1905 the line of the railroad on Wards Island was moved farther north, in order to keep it a greater distance from the State Hospital buildings there. This resulted in a sharp 3° 10′ curve extending at both ends almost to the shore line of the island. A long shore span such as would have been required for a cantilever or suspension bridge would have necessitated a still sharper curve, which was not desirable on account of the heavy grade.

Moreover, in the case of a cantilever or suspension bridge, it would have been necessary, in order to keep the span length down to 850 ft., to place the main piers close to the shore lines. On the Long Island side this would have necessitated, at considerable additional expense, the shifting landward of the boulevard which runs along the river shore, and would undoubtedly have been objected to by the City authorities.

These conditions induced Mr. Lindenthal to investigate a single-span arch bridge design. For an arch, the location of the Long Island abutment on the land side of the boulevard was the proper one, the span length of about 1 000 ft. being determined by the clear height of 135 ft. which was required by the Government for navigation and had to be maintained for the full width between the established bulkhead lines. By that time, also, preliminary borings had been completed, giving more accurate information as to the soil conditions. These borings indicated that solid foundations, which are necessary to resist the tremendous thrust of an arch of such great span, could be had at reasonable depth.

Comparison of Arch with Cantilever and Suspension Types.

Comparison in cost with the suspension and cantilever designs indicated a saving in favor of the arch. The estimated weight of steelwork of the arch, using carbon steel only, was about 13 000 tons, as

compared with 14 000 and 17 000 tons for the suspension and cantilever designs, respectively. As actually built, the arch is probably no cheaper than a cantilever or suspension bridge under the same conditions, because the saving in steelwork was practically offset by the greater cost of the more elaborate towers adopted in the final design, and by the greater cost of foundations, due to the unfavorable soil conditions encountered on the Wards Island side.

Had the arch abutments been designed to satisfy only the static requirements, a substantial saving in favor of the arch would have resulted, and, of course, still more so, if it had been possible to give the arch a span of only 850 ft. These conditions, together with the considerations for appearance, finally led to the adoption of the arch type.

The two masonry towers placed at each end of the bridge are an architectural necessity. Without them, the arch would lose much of its monumental character and be reduced to a utilitarian structure similar to the cantilever. The towers, however, have their static function also. With their great weight, they steepen the resultant arch thrust and thereby limit the size of the deep foundations to a minimum. They also facilitated and cheapened the erection of the arch to a considerable extent. Aside from the considerations which favored it in this case, the arch type, as finally adopted, possesses over the suspension and cantilever types the advantages of greater rigidity, its vertical deflection under live load being only about two-thirds of that of the cantilever. The deflections, which are maximum at the quarter-points of the arch span, are not greater than those at the center of a simple span, which is well known as the stiffest type of bridge.

The secondary stresses, also, are small in an arch of the adopted type, and very much smaller than in most cantilever bridges, although this depends very much on the truss system and the method of erection.

The economy of an arch depends largely on the method of erection. An arch is erected either on falsework (which was out of question in this case) or by the cantilever method, being, in the latter case, held by the temporary back-stays. These back-stays may require considerable extra material, and may thus impair the economy of the arch, unless the configuration of the ground is such that they can be made of short ties and anchored cheaply in solid rock.

This was not possible in the case of the Hell Gate Bridge. However, the adjoining viaduct spans, the floor system, suspenders, and

other parts of the arch bridge proper afforded in this case ample material to make up the temporary back-stays and anchorages, so that only little extra material had to be used. This cheapened the erection very considerably. The erection on the cantilever principle presents no more serious problems than that of a cantilever proper; on the contrary, the final erection adjustments are simpler in the case of the arch.

Two Comparative Designs of Arch Type.

Two designs for an arch bridge were made by Mr. Lindenthal in 1905. Fig. 6 represents the two-hinged crescent arch, as used, for instance, for the railroad bridges over the Garabit Valley, in France, and over the Douro River, in Portugal, with spans of 541 and 525 ft., respectively. Fig. 7 represents the two-hinged, spandrel-braced arch, similar to the bridges over the Rhine at Düsseldorf and Bonn, which have spans of 595 and 614 ft., respectively. The two designs were made for the same loads and specifications as the cantilever or suspension designs, but ordinary carbon steel was assumed throughout, because, at prevailing prices, nickel steel ($40 per ton higher than carbon steel) did not seem to offer any saving. The estimated weight of steelwork was slightly in favor of the crescent arch, but the spandrel-braced arch offered greater advantages for the cantilever erection. Although both designs are pleasing in appearance, the spandrel arch, owing to its height increasing from the center toward the ends, is more expressive of rigidity than the crescent arch, the ends of which appear to be unnaturally slim in comparison with the great height at the center.

The three-hinged arch was not considered. It is not cheaper than the two-hinged arch, and is not as rigid. The fact that the two-hinged arch is statically indeterminate is frequently cited as an objection to that type. This is not justified. There is no more uncertainty in the stress distribution in that type than in a so-called statically determinate structure with riveted connections, and even pin connections do not remove the uncertainty, as is now well recognized. Moreover, if desired, it is always possible, as has been done in the case of the Hell Gate Bridge, to erect the two-hinged arch so that it is statically determinate for dead load. This, however, is a convenience in erection rather than an advantage as regards stress action. Of course, the above characteristic of the two-hinged arch is a serious objection where no solid foundations can be obtained, because of the uncertainty of

stresses which may be produced by a spreading of the foundations. A two-hinged arch requires unyielding abutments.

Modifications in Adopted Arch Design.

The spandrel-braced arch type was finally adopted. Before the design was worked out in detail, the top chord was changed by giving it a slight reversal of curve toward the ends, in order to provide for a stout portal and wind bracing for the top chords, and also improve the silhouette of the arch. The towers were increased in height, and their architectural features modified.

In 1907, the design of the tower shown in Fig. 8 was submitted to the Municipal Art Commission of New York. This Commission, although not objecting to the design as a whole, disapproved of the decorative features of the towers and their bases. The towers then received several further modifications until, in 1912, shortly before construction started, the design illustrated in Fig. 9 was finally adopted. The tower in this design represents a great improvement over that shown in Fig. 8, being more impressive in outline and simpler in architectural ornament and, therefore, more in harmony with the simple, imposing forms and lines of the bridge proper.

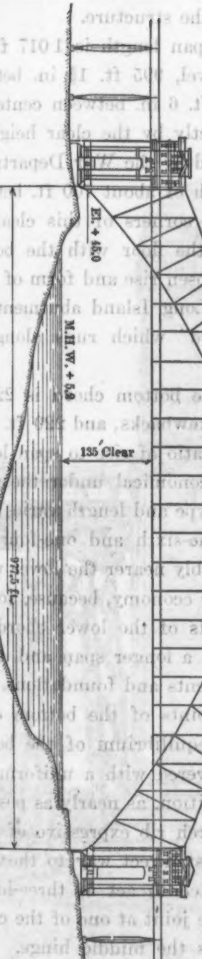
It should also be mentioned that, in 1910, the steel superstructure was re-designed to carry Cooper's E-60 loading and a solid ballasted floor. This loading had already been adopted by a number of railroads, including the New York, New Haven and Hartford Railroad, which is to use the bridge. High-carbon steel was adopted in place of the ordinary structural steel. This last design, moreover, was based on special rules of design prepared by Mr. Lindenthal, as abstracted hereinafter. These resulted in a heavier floor system, a stronger connection, and heavier details throughout. Four lines of stringers and floor-beam brackets, strong enough to carry trolley traffic, were provided outside of the trusses. These modifications increased the total weight of the steelwork to 18 900 tons.

4.—General Arrangement, Proportions, and Cost of Arch Bridge.

The Hell Gate Bridge, as built (Plate XXV), is a two-hinged spandrel-braced arch, carrying four tracks. Its general proportions were dictated partly by local conditions and partly by the requirements for economy and rigidity. The artistic outlines of the steel

CRESCENT ARCH DESIGN (1905)

FIG. 6.



SPANDREL BRACED ARCH DESIGN (1905)

FIG. 7.

superstructure are the result of the proper interpretation of the economic and engineering requirements of the structure.

Span Length and Rise of Arch.—The span length is 1 017 ft. between faces of masonry towers at track level, 995 ft. 1⅜ in. between centers of bearings on skewbacks, and 977 ft. 6 in. between centers of hinges. The length was determined indirectly by the clear height of 135 ft. above mean high water, as prescribed by the War Department, and which had to be maintained for a width of about 700 ft. between the established bulkhead lines. The upper corners of this clearance rectangle fixed the intersection points of the floor with the bottom chord, and thus, in combination with the chosen rise and form of arch, fixed the location of the abutments. The Long Island abutment had to be placed so as to clear the "boulevard" which runs along the shore line.

The exact rise of the center line of the bottom chord is 228 ft. 9$\frac{11}{16}$ in. above the centers of bearings on skewbacks, and 220 ft. 0 in. above the centers of hinges, which gives a ratio of rise to span length of 1 : 4.5. This ratio is about the most economical under the given conditions. The weight of an arch of this type and length varies inappreciably for variations in rise between one-sixth and one-fourth of the span, the absolute minimum being probably nearer the lower value. The greater rise in this case secured greater economy, because, for the given length between the intersection points of the lower chord and the floor, a flatter arch would have required a longer span and, therefore, more metal and more expensive abutments and foundations.

Shape of Bottom Chord.—The panel points of the bottom chord lie on a parabola, this being the line of equilibrium of the bottom chord as an independent arch rib, when covered with a uniform load over the whole span. To approach this condition, as nearly as possible, and thus secure for each truss a massive arch rib expressive of great strength, and transmit the loads in the most direct way to the abutments, the trusses were designed and erected to act as three-hinged arches for the entire weight of steelwork, the joint at one of the center panel points of the bottom chord acting as the middle hinge. This arrangement is conducive to economy and rigidity.

Height of Arch Trusses, and Shape of Top Chord.—The total height of the arch trusses from center of hinge to center of top chord at the middle of the span is 260 ft. 2⅝ in., and the total height of the steel

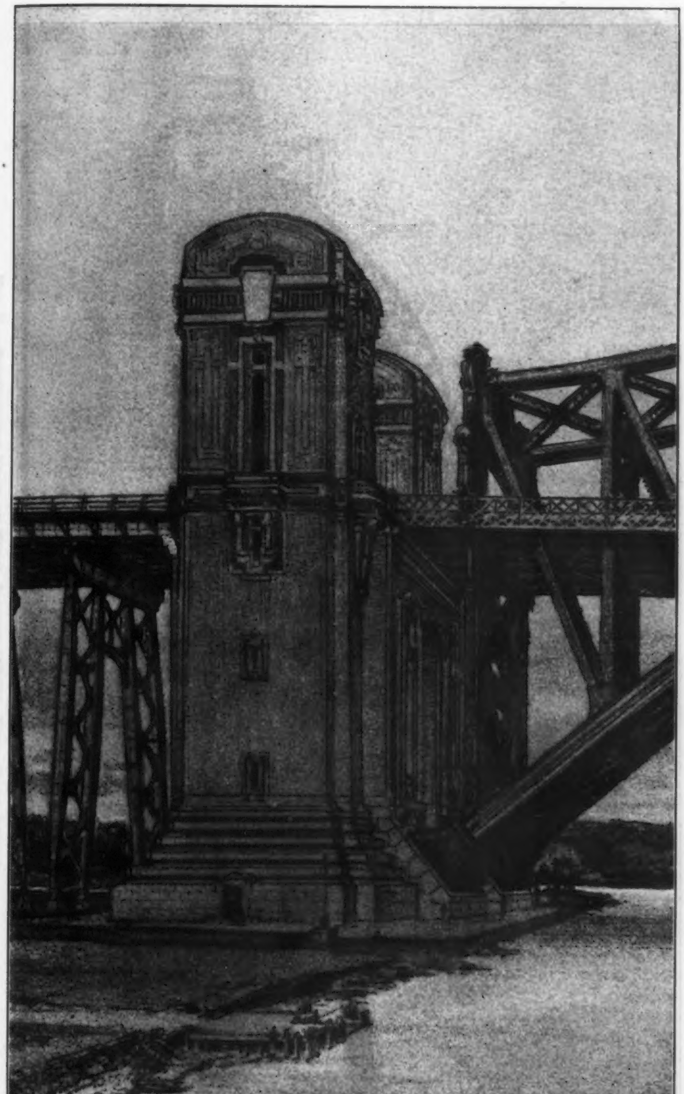


FIG. 8.—PERSPECTIVE VIEW OF ARCH DESIGN (1906), HELL GATE BRIDGE.

Fig. 9.—Perspective View of Final Design (1912), Hell Gate Bridge.

FIG. 9.—PERSPECTIVE VIEW OF FINAL DESIGN (1912), HELL GATE BRIDGE.

PLATE XXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1417.
AMMANN ON
HELL GATE BRIDGE.

GENERAL PLAN
OF
HELL GATE BRIDGE
AND
PORTIONS OF
WARDS AND LONG ISLAND VIADUCTS

superstructure above mean high water is 305 ft. The height of the trusses at the quarter points of the span, where the greatest deflections occur, was chosen at 60 ft., or slightly more than one-fourth of the rise of the bottom chord. This proportion of height to rise insures ample rigidity. Besides, it is sufficient to keep the maximum live-load stresses in the bottom chord approximately within the stresses from live load covering the whole span, and, therefore, no extra material is required in the bottom chord for the otherwise large stresses from partial live load.

On the other hand, for a greater proportion of height to rise than that chosen, the aggregate weight of the top chord and web members increases, and the weight of the bottom chord remains nearly constant.

The height of the trusses was decreased toward the center to 40 ft. 2$\frac{3}{8}$ in., or approximately one-twenty-fourth of the span length, so as to reduce the temperature stresses, which are greatest at the center. The height of 140 ft. at the ends of the trusses was determined by the necessity for rigid portals between the end posts above the track floor. These assumptions for height resulted in the slightly reversed curve of the top chord, which produces a very pleasing sky line.

Width.—The width of 60 ft. between centers of trusses resulted from the required clear width of 53 ft. for the four tracks (the distance between centers of tracks being 13 ft.) and an allowance of 7 ft. for the width of the bottom chord at its intersection with the floor. This width, being one-sixteenth of the span length, was sufficient for lateral stability and rigidity, and therefore it was not necessary to spread the trusses or to place them in inclined planes. To obtain great lateral rigidity of the suspended floor and an economical floor wind truss, however, the latter is made 93 ft. wide, its chords being placed 16$\frac{1}{2}$ ft. outside of the main trusses and carried by cantilever extensions of the floor-beams.

Web System and Panel Length.—The web system consists of a single line of verticals and diagonals, the latter falling toward the center of the span, as commonly used in arch bridges of this type. The system is simple, economical, and free from large secondary stresses. There are twenty-three equal panels, each 42$\frac{1}{2}$ ft. long. For a two-hinged arch, an odd number of panels produces a better appearance than an even number. The panel length was chosen with a view to obtain the most economical floor and truss web system. The latter was secured by

making the average inclination of the diagonals about 45° with the horizontal.

General Arrangement of Arch Bracing.—The transverse bracing between the two trusses comprises a lateral system along the top chords, a lateral system along the bottom chords, and sway-frames and portals in the planes of the first five verticals at each end of the span. Sway-frames between the other truss verticals and between the floor suspenders have been omitted purposely, as they are not needed and would have to be very heavy to resist the stresses from unequal deflection of the two trusses under one-sided load. The top lateral truss is assumed to transmit its reactions to the portals between the end posts and through these and the sway-frames below the floor to the bearings. The wind forces which act along the bottom chords are transmitted through the bottom lateral truss directly to the arch bearings. Owing to the polygonal shape of the chords, components of the lateral stresses are transmitted into the main trusses at each panel point, which, although small, had to be considered in proportioning the truss members. At the intersections of the floor with the bottom chord, the lateral bracing between these chords had to be interrupted to provide the necessary head-room above the floor. Stiff portals were substituted for the laterals at these points, and the chords were proportioned for the additional bending stresses.

General Arrangement of Floor System.—The floor system (Plates XXVIII and XXXII) comprises the following parts:

1.—A floor-beam at each panel point, rigidly framed into the trusses at the first four verticals at each end of the span and hung from the trusses by suspenders in the middle portion of the span.

2.—Eight lines of railroad stringers, 6 ft. 6 in. apart, framed into the floor-beams and braced together in pairs for each track by top and bottom laterals and sway-frames. Each pair carries a concrete trough which supports the ballasted track.

3.—Four lines of sidewalk stringers, one pair outside of each track, framed into cantilever extensions of the floor-beams. These stringers support only a light sidewalk, but are made strong enough to carry the trolley line which was contemplated.

4.—Two lines of lattice girders, one on each side of the floor, placed 16½ ft. outside of the center line of the main truss, and carried at the end of the floor-beam extensions. The function of these girders is to

screen the floor system and thus secure a more uniform and neat appearance. At the same time, these girders act as railings, and their bottom chords form the chords of the floor lateral truss.

5.—A floor lateral truss to resist the wind and lateral forces which act on the trains and floor.

6.—Two "braking girders", one at each intersection of the floor with the main trusses. These girders transmit the longitudinal forces from braking and traction from the stringers to the main trusses, and thus eliminate serious horizontal bending of the floor-beams.

Provision for Expansion of Floor.—The floor had to have at least one expansion joint at or between its intersections with the bottom chord (Panel Points 6, Plates XXV and XXVIII), so as not to be strained by temperature changes or deformation of the arch trusses.

The expansion of the floor for a change in temperature of $\pm$ 72° Fahr. is $\pm$ 4.1 in., but this is partly offset by an increase of $\pm$ 1.6 in. in the distance between Points 6, due to the temperature deformation of the arch truss, leaving a movement from the normal position, of $\pm 4.1 \mp 1.6 = \pm 2.5$ in., which had to be provided for at the expansion joint. The effect of a maximum live load covering the entire span is to open the joint by 0.1 in., which is negligible.

In deciding on the location of the expansion joint, the following conditions had to be taken into consideration:

First.—To secure the greatest lateral rigidity, the floor lateral system should be such as to cause the least lateral deflections.

Second.—To avoid large stresses in the stringers and their connections from the longitudinal force, the distance between the expansion joint and the braking girder should be as small as possible.

Third.—The floor suspenders should be subject to the least possible bending in the plane of the truss from the expansion of the floor.

To meet all these conditions, the expansion joint was placed at Panel Point 12, six panels from the Wards Island end. At the corresponding Point 12 on the Long Island side, the floor laterals are rigidly connected at the center of the floor-beam, but the wind chords are cut, so as to secure hinge action of the floor-lateral truss. The latter, therefore, forms a three-span cantilever truss with a suspended span between Points 12, Cantilever Arms 12—6, and Anchor Arms 6—0.

The suspended span delivers its reactions to the cantilever arms by a fixed connection of the diagonals at the center of Floor-beam 12 (Long Island side) and a sliding bearing at the center of Floor-beam 12 (Wards Island side). The reactions at Points 6 are transmitted to the bottom chord lateral system and, through this, to the arch bearings. The reactions of the floor-lateral truss at the ends are transmitted to the sway-frames between the end posts and, through these, to the bearings.

The longitudinal force from Part 0—12 (Wards Island side) is transmitted to the braking girder at 6 (Wards Island side), and the force from Part 12 (Wards Island side) to 0 (Long Island side) is transmitted to the braking girder at 6 (Long Island side). The stringer connections to the floor-beams are made strong enough to resist safely the longitudinal force in addition to the vertical shear.

Cost.—The Hell Gate Bridge contains approximately 110 000 cu. yd. of masonry in the towers and foundations and 19 400 tons of steel in the steel superstructure. (Detailed quantities are given under the respective headings.)

The cost of construction is approximately as follows:

Towers and foundations	$1 700 000
Steel superstructure	1 900 000
Concrete flooring and tracks	100 000
Total	$3 700 000

5.—Masonry Towers and Foundations.

The massive masonry towers which flank the steel arch greatly enhance the appearance of the bridge and give it its monumental character. They also give expression to the solidity of the abutments to resist the great thrust of the arch. Without the towers, the statically trained eye would want that expression of stability, because of the comparative flatness of the shores.

This static requirement, however, is not merely an apparent one. Preliminary wash-borings indicated that the foundations had to go to considerable depth, at least on the Wards Island side. There having been doubt as to the reliability of the wash-borings, the depth of rock was established later by core-borings at from 55 to 140 ft. below mean high water line. To restrict the size of the foundation to a minimum,

PLATE XXVI.
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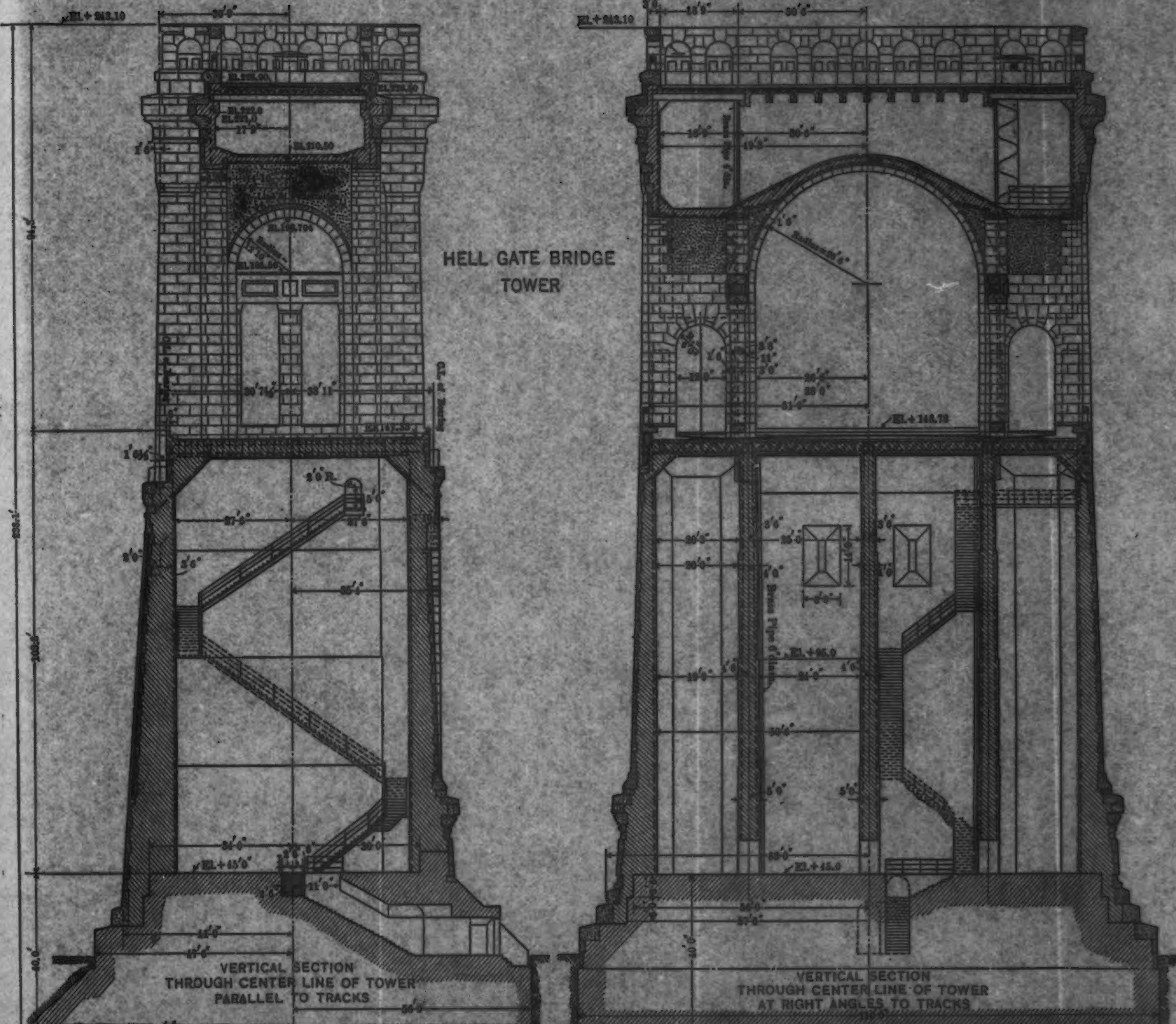

it was necessary to provide above the ground a mass of masonry, the weight of which, combined with the inclined reaction of the arch, would give a steep resultant, passing well within the middle third of the foundation area, so that the edge pressure could be kept within permissible limits.

To be expressive of their purpose, the towers were designed architecturally as massive masonry blocks with simple outlines and plain structural ornamentation (Fig. 10). In working out the architectural form and details of the towers, Mr. Lindenthal had the valuable assistance and advice of Mr. Henry Hornbostel as Consulting Architect.

Towers Above Foundations.—Plate XXVI shows the type of construction of the towers above the foundations. The dimensions of the towers are 103 by 139 ft. at the ground level and diminish along parabolic lines to 61 by 105 ft. at the top. The total height above ground is 220 ft., and the extreme height above bottom of foundation is 345 ft. Each tower has a solid base, which acts as an abutment for the arch bearings and distributes the pressure over the foundations. On the Long Island side, the base, with the foundation course, forms a monolithic slab, 140 by 104 ft., with an average thickness of 49 ft., and rests on gneiss bed-rock, which was encountered at from 15 to 38 ft. below the surface, and was reached by open excavation. The maximum foundation pressure is $8\frac{1}{4}$ tons per sq. ft. On the Wards Island side, the base is 140 by 119 ft. and 40 ft. thick, and rests on the caisson foundation described subsequently.

Above this base, and up to the track floor, the towers are of hollow cellular construction, consisting of the four exterior walls and three interior walls parallel to the tracks. Above the track floor, the transverse walls are pierced by a main arch over the four tracks, and two smaller side arches over the footwalks. The longitudinal walls have also an arch opening for architectural reasons. The towers are topped by a flat roof surrounded by an ornamental balustrade. Stairs lead from an entrance at the ground level through the base and interior vaults to the track floor and roof, and also to the ends of the top chords of the steel arch. The towers are of concrete with granite facing. The concrete is well reinforced with vertical and horizontal steel rods in order to prevent temperature and shrinkage cracks. The track floor and roof are heavily reinforced with steel girders. The Snare and

Triest Company was the contractor for the towers above the bases, and the Ryan Construction Corporation for the bases.

Wards Island Tower Foundation.—On the Wards Island side, the tower base rests on twenty-one concrete caissons, all sunk by the pneumatic process to depths varying from 37 to 107 ft. below mean high water, which is 20 ft. below the ground surface. (Plate XXVII.) The caissons are arranged in five rows parallel to the axis of the bridge. Each outer row and the middle one consists of five cylindrical caissons, 18 ft. in diameter, which are calculated to carry only vertical pressure. Each of the two other rows consists of three rectangular caissons, 30 by 41 ft., which are interlocked by concrete keys extending nearly the full depth of the caissons. These two rows of caissons thus form two rectangular blocks, 30 by 125 ft., which are calculated to resist entirely the horizontal pressure from the arch. They exert a maximum edge pressure on the rock foundation of 20 tons per sq. ft., if skin friction and buoyancy are neglected. The space for the keys was excavated and filled with concrete, partly with and partly without the use of air pressure, after the caissons had been sunk to their final depth.

The dissection of the foundation into twenty-one individual caissons was advisable on account of the large size of the tower base and the greatly varying depth to solid rock. The formation of the bed-rock below Wards Island is very peculiar. One of the lines of cleavage between the dolomitic limestone formation and the gneiss rock which run parallel with the East River, appears to pass right under the Wards Island tower. About 45% of the foundation on the river side is on limestone and about 25% on the land side is on gneiss. Between these two rock formations there is a crevasse of unknown depth and of from 15 to 60 ft. width. Its sides are almost vertical, corresponding to the vertical stratification of the rock. The crevasse is filled mostly with red clay and some boulders. The clay is practically impervious to water, which is shown by the fact that the excavation was in part carried on with an air pressure of only 18 lb. at a depth of 100 ft. below water level, and, for some caissons, to 18 ft. below the cutting edge. In its natural state, the clay is very hard and has a high bearing capacity, but, in water, it dissolves readily.

Three of the fifteen cylinder caissons rest entirely on this clay, at depths of from 94 to 123 ft. below the surface, where there is no danger of disturbance. Under the rectangular caissons, the crevasse was

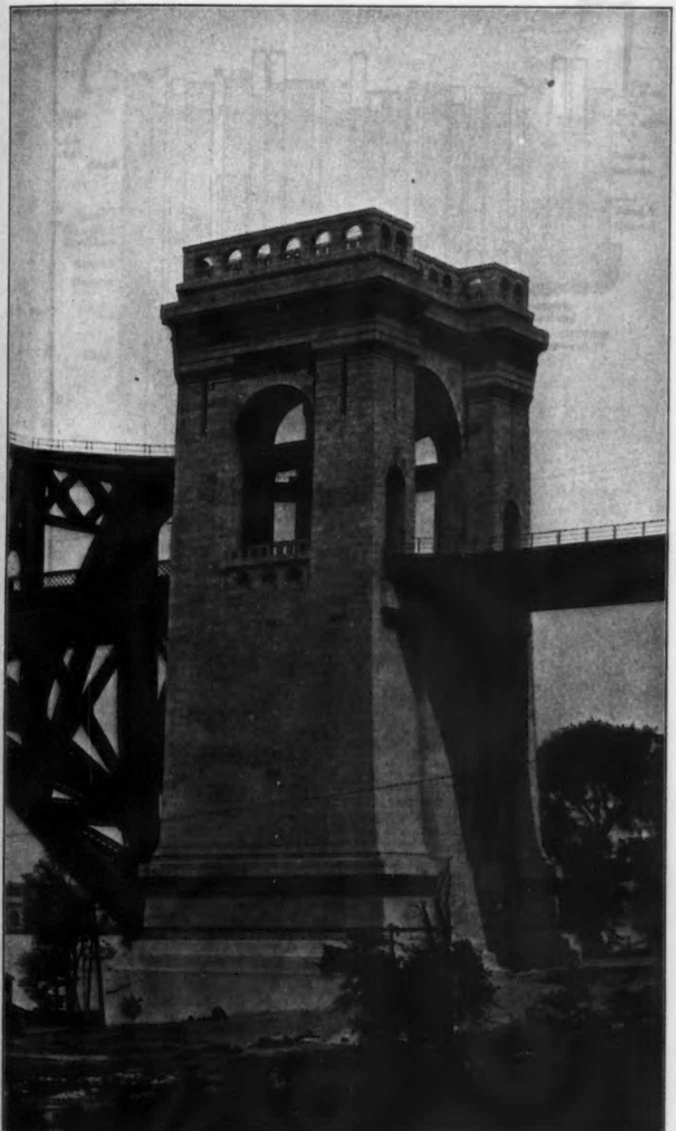

Fig. 10.—Long Island Tower of Hell Gate Bridge.

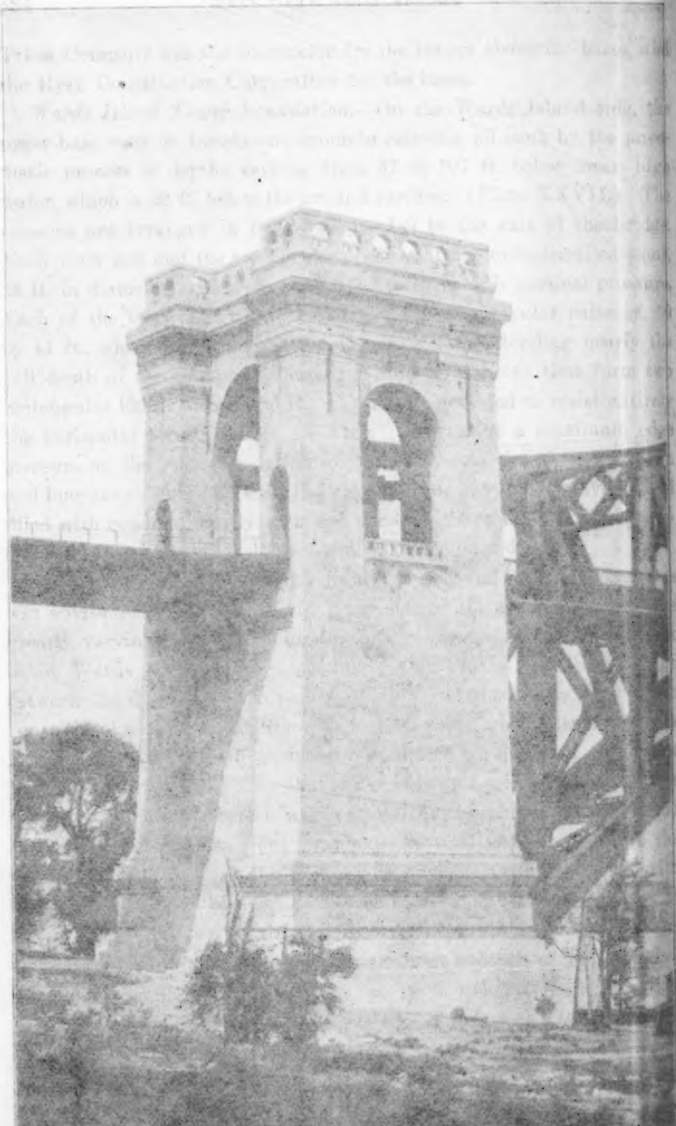

FIG. 10.—LONG ISLAND TOWER OF HELL GATE BRIDGE.

PLATE XXVII.
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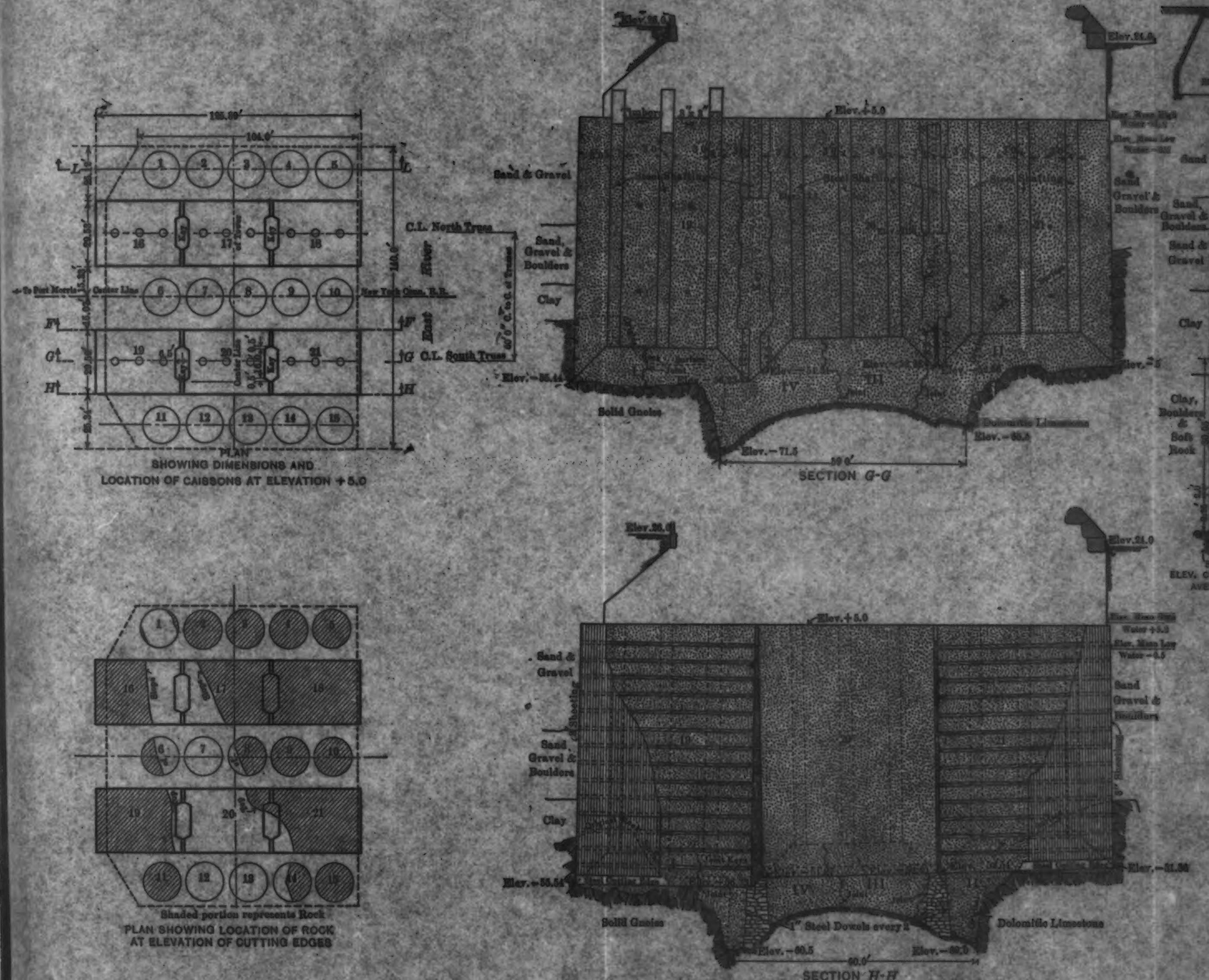
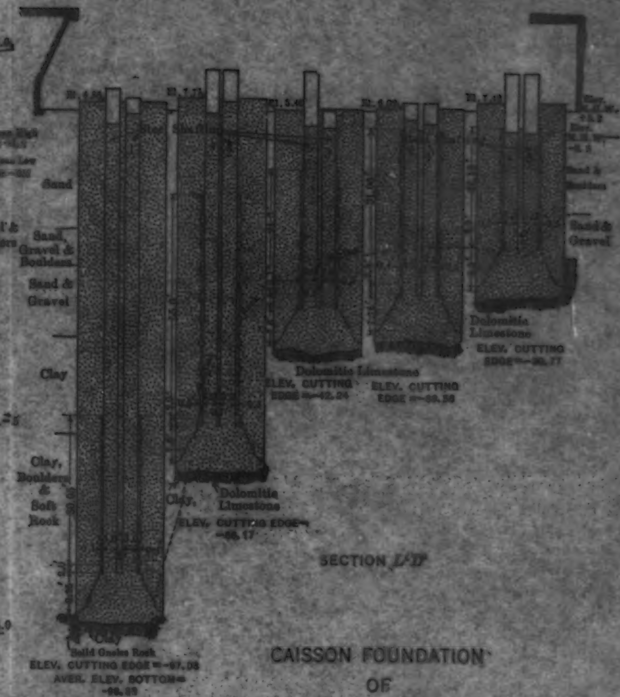




CAISSON FOUNDATION OF WARDS ISLAND TOWER

NOTES:

Caissons shown on this plate, were sunk in the following order; 20 19-21
Rubble masonry piers built under E. & W. Cutting edges of Caissons 19 & 20 at N. & S. sides and under 20 & 21 at S. side only. These piers were constructed to support caissons during excavation for Arch, and were left in place.

Order of procedure in excavating and concreting working chamber.
1st — Part I, excavated and concreted continuously
2nd — " II " " " "
3rd — " III " " " "
4th — " IV " " " "

Keyways between caissons 19 & 20 and 20 & 21 excavated and sealed under air pressure.
Total linear feet of sinkage = [illegible]
Steel shafting left in place.
Average air pressure during arch construction = [illegible] lb.
" " " " sinking = 30 lb.
Concrete in all Caissons: 1 part Portland Cement, 2 parts Sand, 4 parts Broken Stone. (Large Rubble Stone embedded in concrete)
Datum; All elevations given, refer to datum of New York Connecting R.R., which is equal to Mean Low Water at Battery = Elev. 0.0

bridged over by a concrete arch which was built at considerable risk below the cutting edge of the caissons. For this purpose, the caissons were supported temporarily by pilasters of rubble masonry built down to the intrados of the arch previous to the excavation for the latter (Plate XXVII). All caissons are of 1:2:4 concrete, well reinforced horizontally and vertically with steel rods. Details of the working chambers and cutting edge are shown in Fig. 11.

Owing to the unusual and difficult character of the Wards Island foundation, the Company decided to do the work with its own forces, under the direction of the Chief Engineer. P. G. Brown, M. Am. Soc. C. E., as Managing Engineer, was in immediate charge of this work.

Quantities.—The two towers contain approximately 110 000 cu. yd. of masonry, of which 28 000 cu. yd. are in the Wards Island foundation. The principal quantities are:

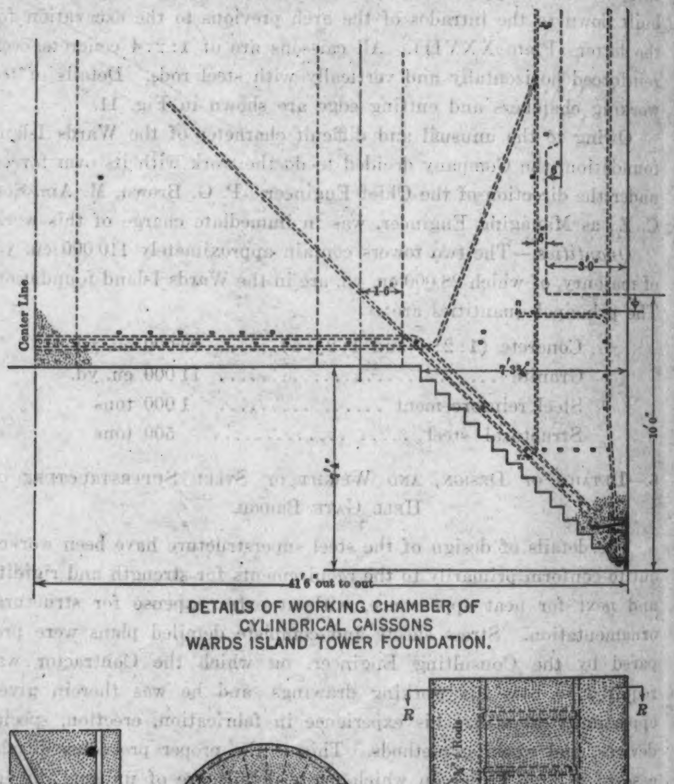
Concrete (1:2:4 and 1:2½:5)......	99 000 cu. yd.
Granite	11 000 cu. yd.
Steel reinforcement	1 000 tons
Structural steel	500 tons

6.—Details of Design, and Weight of Steel Superstructure of Hell Gate Bridge.

The details of design of the steel superstructure have been worked out to conform primarily to the requirements for strength and rigidity and next for neat appearance, without extra expense for structural ornamentation. Stress sheets and complete detailed plans were prepared by the Consulting Engineer, on which the Contractor was required to base his working drawings, and he was therein given opportunity to utilize his experience in fabrication, erection, special devices, and working methods. This is the proper procedure in the case of a large bridge in which many details are of unusual dimensions and composition.

Sections of Truss Members.—The make-up of the sections of the truss members was largely governed by the necessity for riveted connections. Pin connections were not considered. They are objectionable in members subject to reversal of stress, as they impair the rigidity and durability of the bridge. Because of the riveted connections, the number of webs of all members was limited to two. Fig. 12 shows the typical sections of the various truss members.

DETAILS OF WORKING CHAMBER OF
RECTANGULAR CAISSONS
WARDS ISLAND TOWER FOUNDATION.



DETAILS OF WORKING CHAMBER OF
CYLINDRICAL CAISSONS
WARDS ISLAND TOWER FOUNDATION.

DETAIL OF CUTTING EDGE SECTION *R-R* SECTION ON CENTER LINE

FIG. 11.

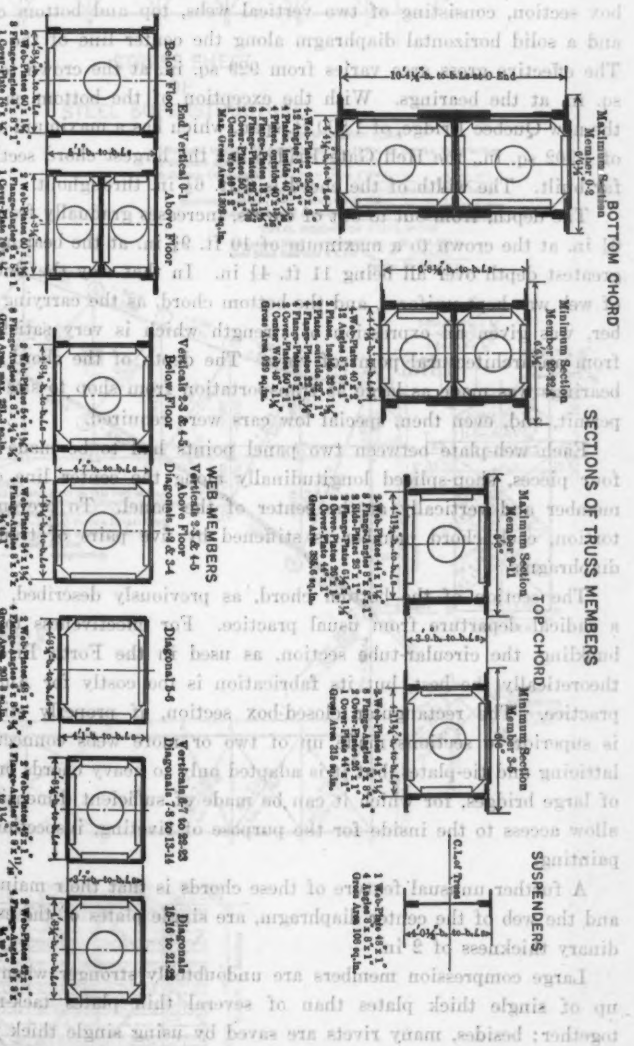

SECTIONS OF TRUSS MEMBERS
BOTTOM CHORD
Maximum Section
Member 0-2
Minimum Section
Member 22-22A
TOP CHORD
Maximum Section
Member 9-11
Minimum Section
Member 3-5
SUSPENDERS
C.L. of Truss
WEB MEMBERS
End Vertical 0-1
Below Floor
Above Floor
Verticals 2-3 & 4-5
Below Floor
Verticals 2-3 & 4-5
Above Floor
Diagonals 1-2 & 3-4
Diagonal 5-6
Verticals 6-7 to 22-23
Diagonals 7-8 to 13-14
Diagonals
15-16 to 21-22

FIG. 12.

Bottom Chord Section.—The bottom chord has a closed double-box section, consisting of two vertical webs, top and bottom covers, and a solid horizontal diaphragm along the center line of the chord. The effective gross area varies from 929 sq. in. at the crown to 1 392 sq. in. at the bearings. With the exception of the bottom chord of the new Quebec Bridge, of 1 800 ft. span, which has a maximum section of 1 902 sq. in., the Hell Gate Bridge has the largest chord section so far built. The width of the chord is 6 ft. 6½ in. throughout.

The depth, from out to out of covers, increases gradually from 7 ft. 0¼ in. at the crown to a maximum of 10 ft. 9¾ in. at the bearings, the greatest depth over all being 11 ft. 4¼ in. In that way the thickness of web was kept uniform, and the bottom chord, as the carrying member, was given an expression of strength which is very satisfactory from an architectural point of view. The depth of the chord at the bearings was made as large as transportation from shop to site would permit, and, even then, special low cars were required.

Each web-plate between two panel points had to be made up of four pieces, shop-spliced longitudinally along the center line of the member and vertically at the center of the panel. To prevent distortion, each chord member is stiffened by five pairs of transverse diaphragms.

The section of the bottom chord, as previously described, marks a radical departure from usual practice. For effectiveness to resist buckling, the circular-tube section, as used in the Forth Bridge, is theoretically the best, but its fabrication is too costly for American practice. The rectangular closed-box section, if properly stiffened, is superior to sections made up of two or more webs connected by latticing and tie-plates, but it is adapted only to heavy chords or posts of large bridges, for which it can be made of sufficient dimensions to allow access to the inside for the purpose of riveting, inspection, and painting.

A further unusual feature of these chords is that their main webs, and the web of the center diaphragm, are single plates of the extraordinary thickness of 2 in.

Large compression members are undoubtedly stronger when made up of single thick plates than of several thin plates tack-riveted together; besides, many rivets are saved by using single thick plates. It is frequently maintained that better material is obtained in the

PLATE XXVIII.
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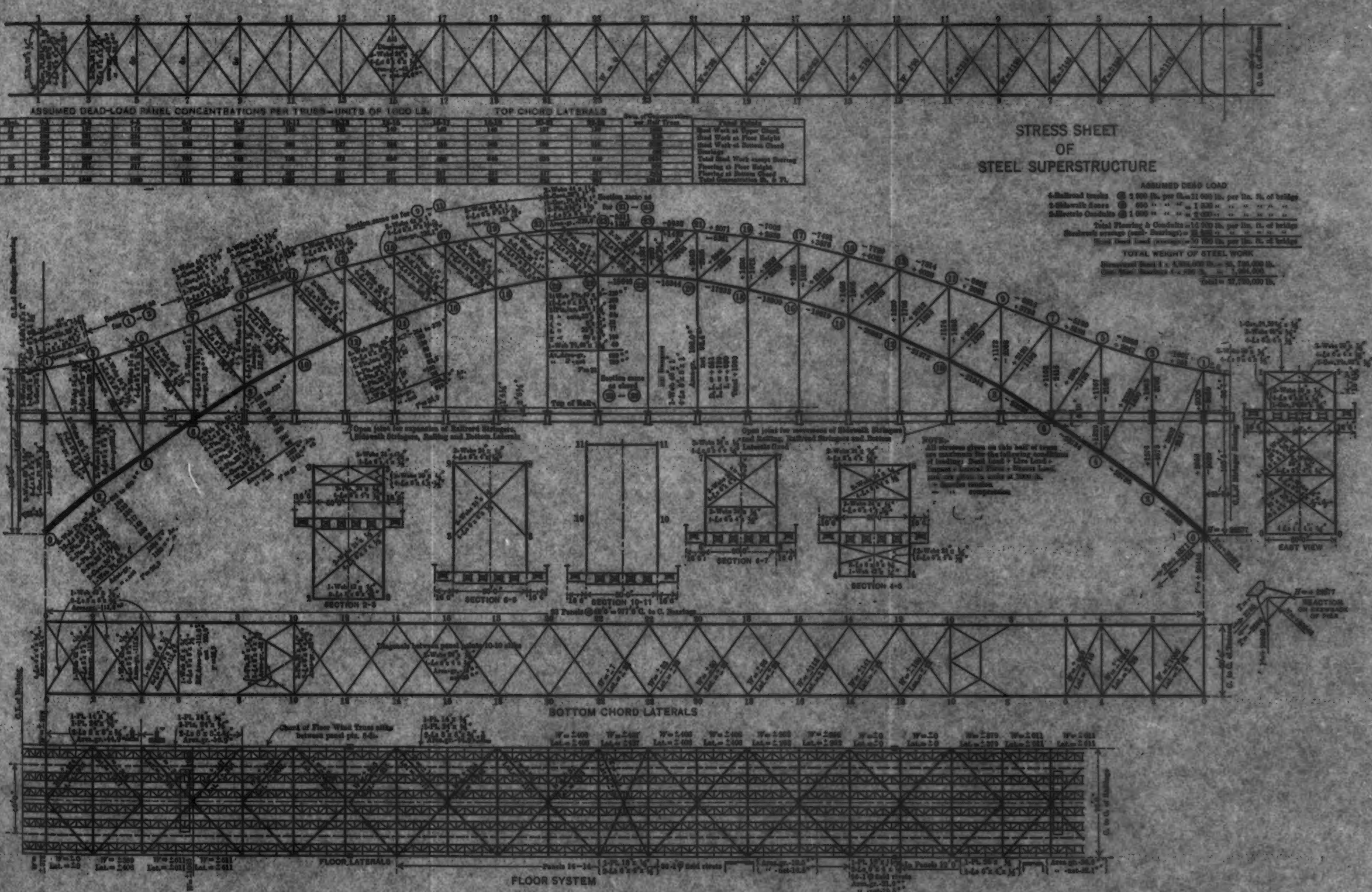

thinner than in the thicker plates. This was not substantiated by the great number of specimen tests made for the Hell Gate Bridge from material varying from ½ in. to 2 in. in thickness. In general, the thick material showed as high elastic properties and ultimate strength as the thinner material rolled from the same heat.

The tests* made by the Society's Special Committee on Steel Columns and Struts show a marked falling off in the compressive strength of heavy columns in comparison with light columns of approximately the same outside dimensions. It would be wrong, however, to conclude that this is due to the thicker metal in the heavy columns. It would seem to be due rather to the less efficient distribution of metal in the heavier columns. If the heavy columns had been built up of several thin plates, tack-riveted together, their compressive strength would probably have been even less. Comparative tests, to throw light on this question, would be highly desirable.

Top Chord and Web Members.—The top chord and web members have a rectangular box section with two solid webs parallel to the plane of the truss. The top chord and end posts are properly provided also with a solid cover. All open sides have stiff-angle latticing (Plates XXIX and XXX). The section of the top chord ranges from 315 to 386 sq. in., and has a uniform depth of 4 ft., except in the end panel, where, for better appearance, and to allow sufficient height for entrance at the tower, the chord tapers to 5½ ft. From the end a stair leads through the chord to its top, along which two light handrails are provided.

The sections of the web members increase from 126 sq. in. at the crown to 315 sq. in. at the end, and the depth increases correspondingly from 42 to 60 in. A good section for compression members of moderate area is that of the two vertical posts, 2-3 and 4-5, below the floor, each flange consisting of two angles. The latticing connects only to the inside angle, but its stresses are transmitted to both angles through short tie-plates placed across the two angles (Plate XXX).

Floor Suspenders.—The suspenders, which carry the floor between its intersections with the trusses, have an I-section with a single web, 48 in. wide, placed at right angles to the plane of the truss. For appearance, and to prevent large bending stresses in the suspenders, due to the longitudinal expansion and contraction of the floor, they

* *Proceedings*, Am. Soc. C. E., for December, 1917.

were purposely made slender in elevation. To prevent bending stresses in the suspenders, due to the vertical deflection of the floor-beams and the horizontal deflection of the floor-lateral truss, the suspenders are connected to the floor-beams at the bottom and to the trusses at the top with 16-in. pins placed parallel to the plane of the truss (Plate XXX).

Latticing of Truss Members.—Since the failure of the Quebec Bridge in 1907, increased attention has been given to the latticing of compression members, which has led to marked improvement in this respect. Numerous tests, and a number of failures that have occurred since, have given further proof that inadequate latticing greatly reduces the buckling strength of compression members. It is now generally recognized that the latticing has a distinct static function, and bears a certain relation to the dimensions, shape, and area of section of the member.

The following simple and easily remembered rule was applied in proportioning the latticing in the Hell Gate Bridge and Approaches:

"The latticing and its connections shall be designed to resist at any section at right angles to the axis of the member a shearing force, in pounds, at least equal to 300 times the gross area of the member, in square inches."

Stiff-angle latticing was used throughout, with a minimum thickness of ½ in., and at least two rivets for each angle. Flat lattice bars in heavy compression members are objectionable, as they have little resistance against compression. They are easily bent in handling the members, and, even if subsequently straightened, as is a too common practice, they constitute a permanent defect in a bridge.

Special attention was given to uniformity and the neat appearance of the latticing. Main truss members, as well as laterals, have double lattice angles, with an inclination of about 45° to the axis of the member. One of each pair of lattice angles is spliced at the intersection by a square plate (Plates XXIX and XXX). All latticing is placed inside the flange angles. The tie-plates are as near as practicable to the end of the member, in any case, well within the edge of the gusset-plate, and all cover-plates of the bottom chord are continuous across the panel point, being notched out for the gusset-plates. This is an important improvement over the very common practice of stop-

PLATE XXIX.
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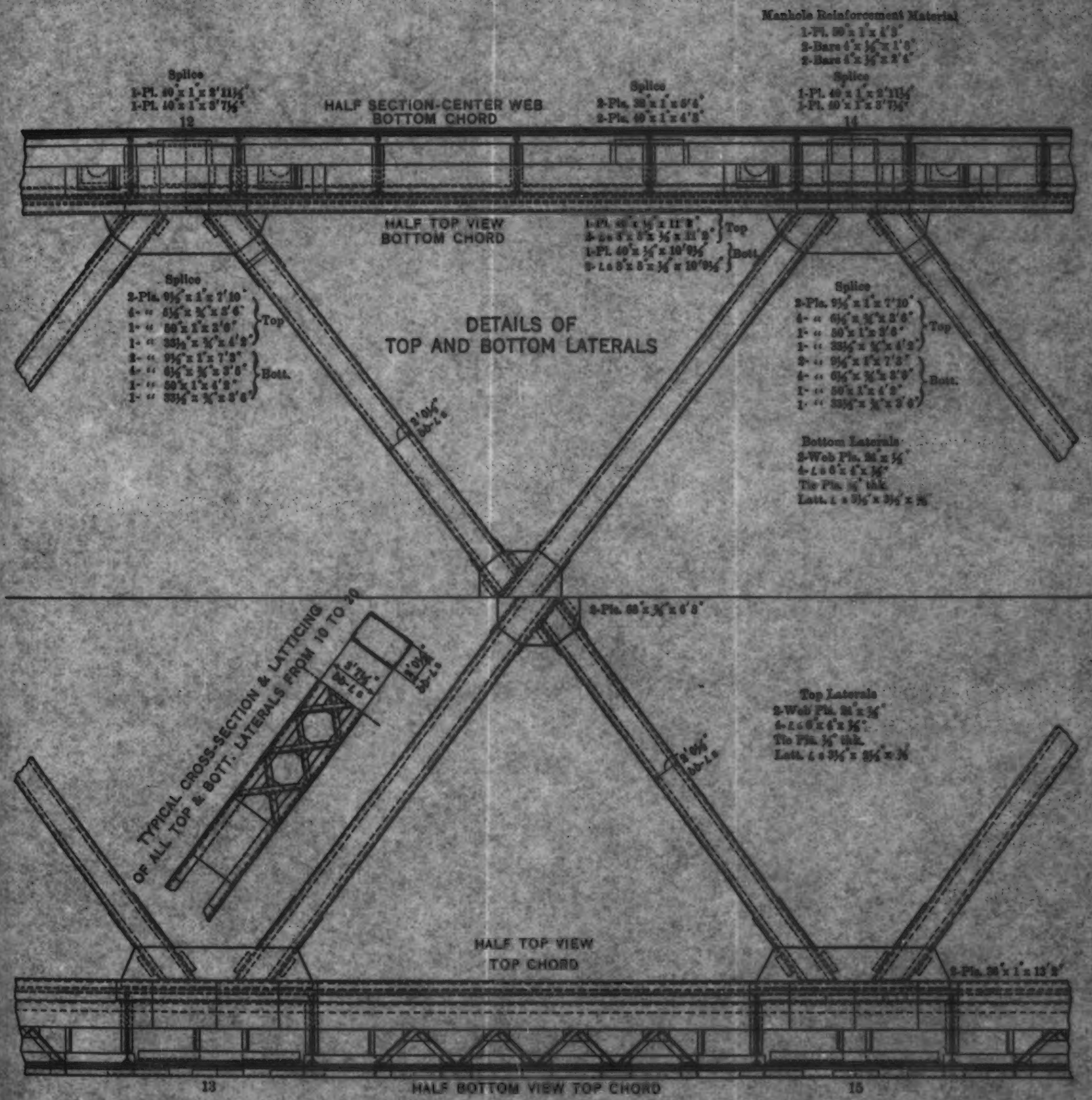


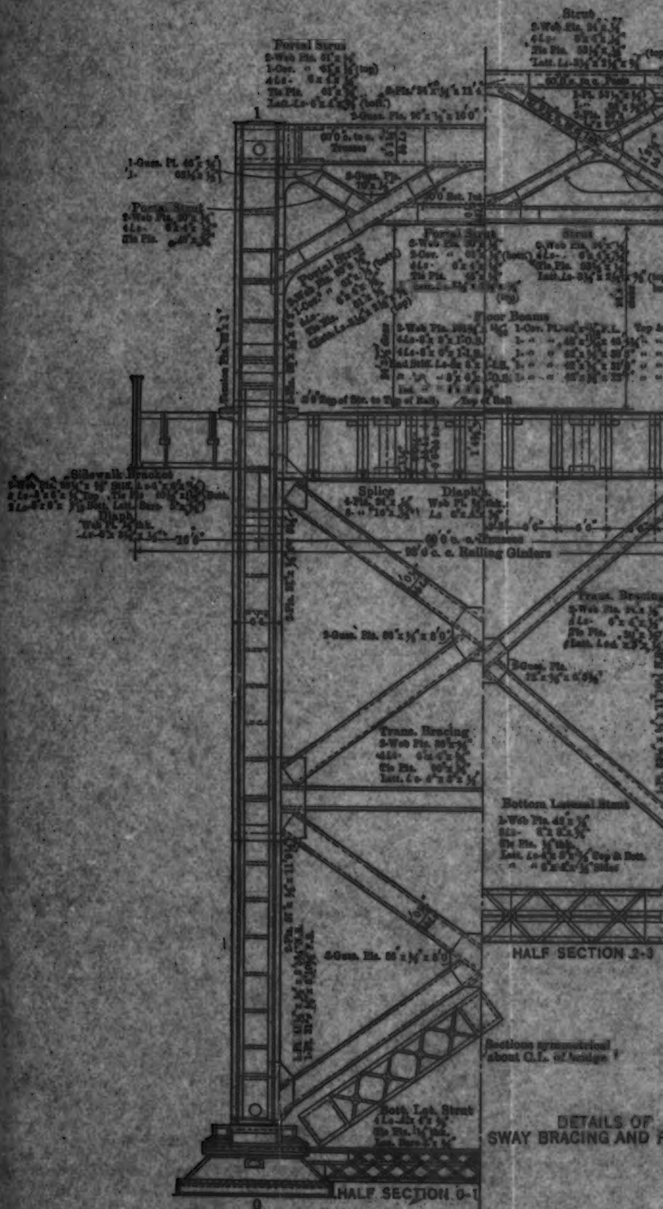
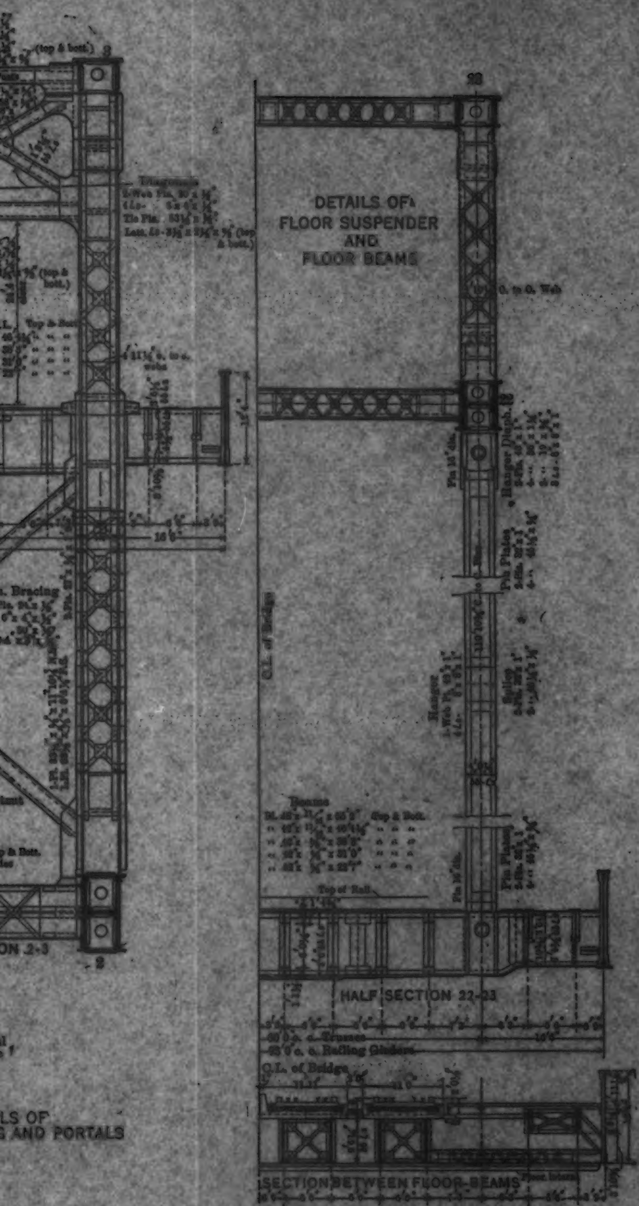

Portal Strut
Strut
Sidewalk Bracket
Splice
Floor Beams
Trans. Bracing
Bottom Lateral Strut
Sections symmetrical about C.L. of bridge
HALF SECTION 2-3
HALF SECTION 0-1
DETAILS OF
SWAY BRACING AND

PLATE XXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1417.
AMMANN ON
HELL GATE BRIDGE.



ping tie-plates or cover-plates outside of the gusset, thus leaving the flange of the member unsupported for a considerable length.

Riveted Connections and Splices of Truss Members.—As yet, little is known about the correct distribution of stresses in a riveted connection. In proportioning, the favorable assumption of uniform distribution of stresses among all rivets of a connection is made, and the secondary stresses in the rivets caused by the stiffness of the connections is generally disregarded, on the assumption that they are fully covered by the margin of safety of the main stresses. In view of the unusually large connections and splices in the main trusses of the Hell Gate Bridge, it was important to guard against local over-stressing.

The following requirements, as quoted from the "Rules of Design," governed the detailing of the connections and splices:

"The strength of connections shall be sufficient to develop the full strength of the member, even though the computed stress is less, the kind of stress to which the member is subjected being considered.

"Truss members with alternating axial stresses caused by live load (including impact) shall be proportioned for the stress requiring the larger sections. The connections and splices shall be proportioned for the larger stress plus 50% of the smaller stress of opposite sign.

"All joints in riveted work, whether in tension or compression, shall be fully spliced, except when differently noted on drawings.

"Rivets carrying calculated stress, and having a grip which exceeds four diameters, shall be increased in number 1% for each additional $\frac{1}{8}$ in. of grip.

"Where splice-plates are not in direct contact with the parts which they connect, the number of rivets on each side of the joint shall be increased over the number theoretically required to the extent of one-third of the number for each intervening plate.

"Rivets carrying strain and passing through fillers shall be increased 100% in number, if the thickness of the filler is equal to the diameter of the rivet, the increase in number being proportionately more or less as the filler shall be more or less thick than the diameter of the rivet."

The further rule was observed to connect, or splice, each individual part of the section of a member as directly as possible, so as to avoid the transfer of stresses through other parts. Typical connections are shown in Figs. 13, 14, 15, and 16.

The connections at each panel point are made with two gusset-plates, 1 in. thick in the middle panels and $1\frac{3}{8}$ in. in the panels nearer the ends. The largest of these are 120 by $1\frac{3}{8}$ in. by 17 ft. 6 in. and

126 by $1\frac{5}{8}$ in. by 14 ft. 6 in., and mark the limits of present rolling mill capacities.

The general scheme in detailing the connections of the web members to the gussets was as follows: The web-plate of the member is in the same plane and has the same thickness as the gusset-plate. It is cut off at the edge of the gusset and spliced to the latter by a splice-plate on each side. The flange angles, however, extend as far as possible over the gusset, their outstanding flanges being connected by lug-

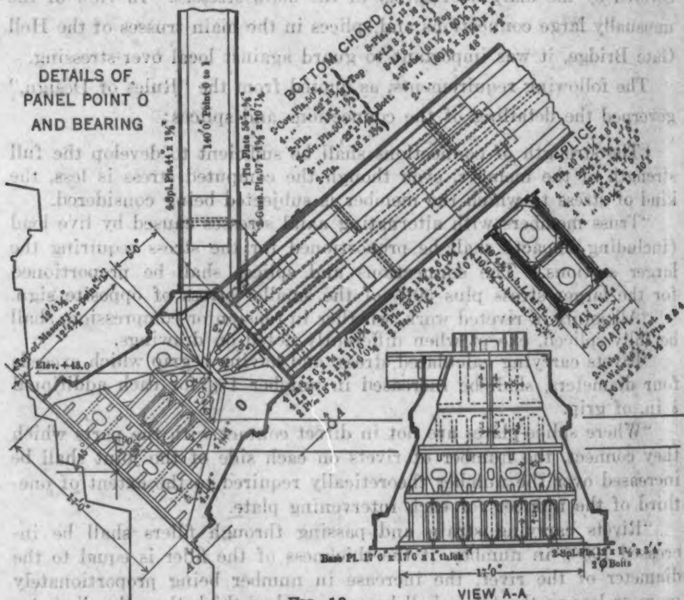


FIG. 13.

angles. This type of connection is efficient and economical, as most of the rivets are in double shear, and the size of the gusset is reduced to a minimum. On account of the re-entrant joints, this connection caused some inconvenience in erection, but no serious difficulties were encountered. The splice-plates were shop-riveted to the member, but, to allow them to spring slightly in entering the member, the two rows of rivets nearest the edge of the gusset-plate were left for field driving.

The top and bottom chords have a butt joint at every panel point on the intersection of the axes of the truss members. The connection

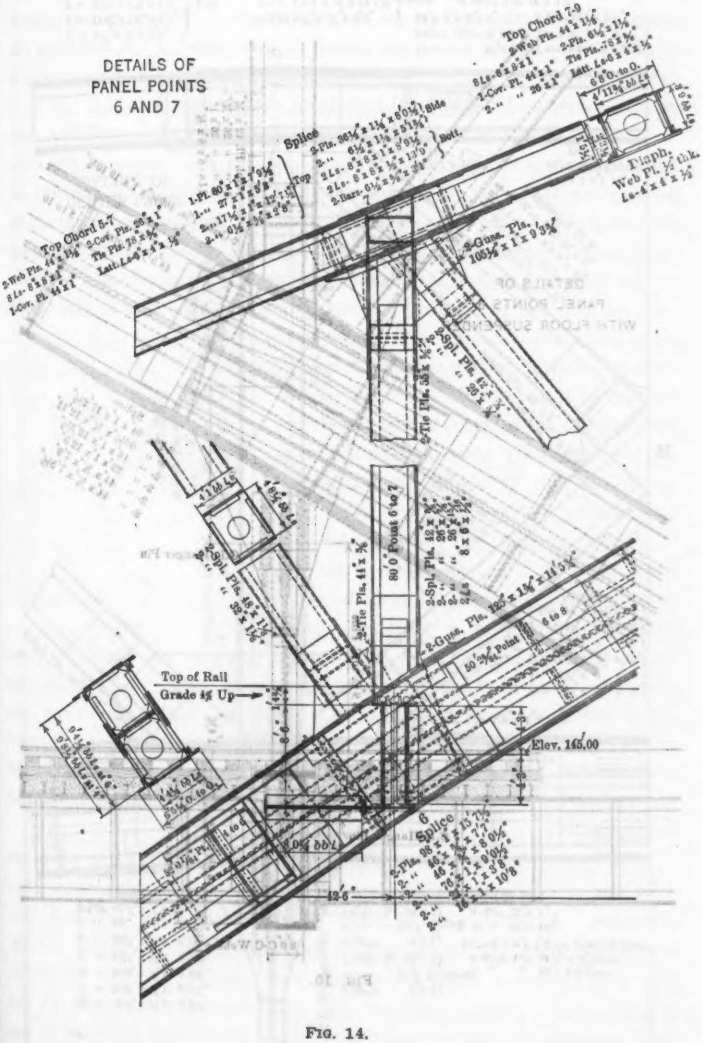

DETAILS OF
PANEL POINTS
6 AND 7
Top Chord 7-9
Top Chord 5-7
Splice
Diaph.
Web Pl. ½ thk.
Top of Rail
Grade 1% Up
Elev. 145.00
80' 0" Point 6 to 7
Splice

FIG. 14.

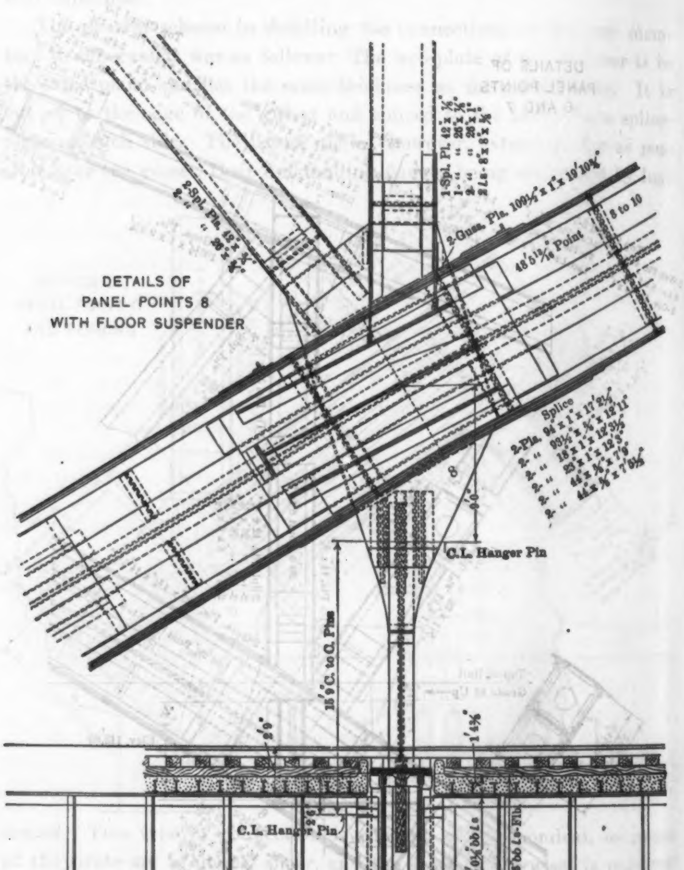

DETAILS OF
PANEL POINTS 8
WITH FLOOR SUSPENDER
2-Guss. Pls. 100½" x 1" x 19' 10⅝"
8 to 10
C.L. Hanger Pin
C.L. Hanger Pin
2' 0" C-C Webs
Splice

FIG. 15.

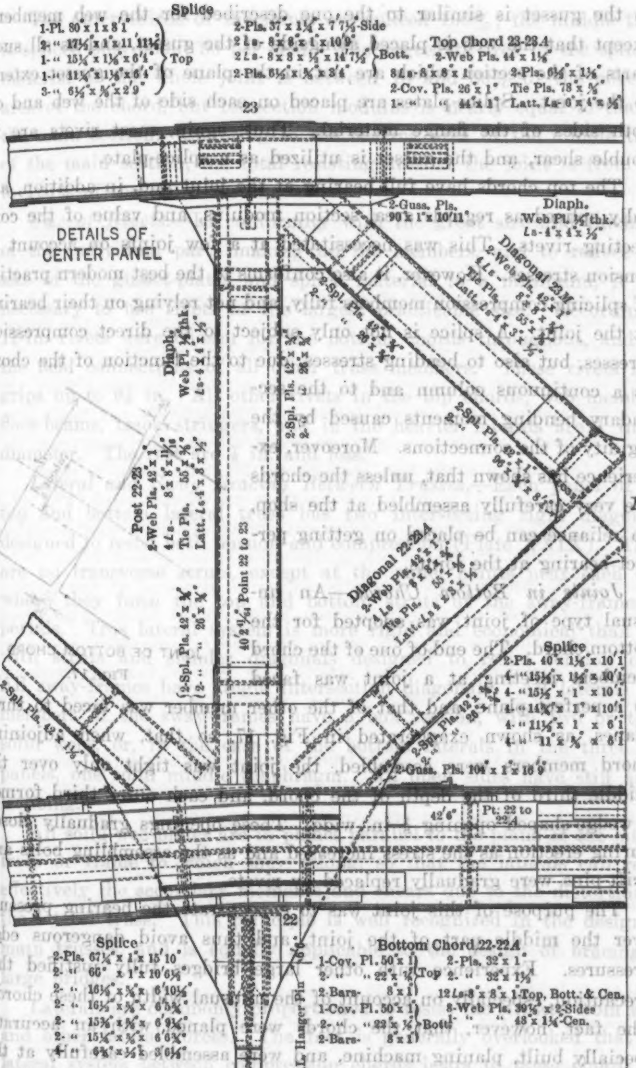
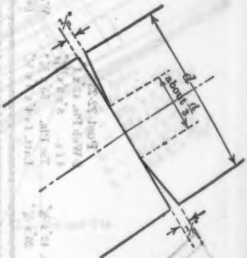

DETAILS OF CENTER PANEL
Splice
1-Pl. 80" x 1" x 8' 1"
2- " 17½" x 1" x 11' 11⅝"
1- " 15¼" x 1⅛" x 6' 4"
1- " 11¼" x 1⅛" x 6' 4"
3- " 6½" x ⅝" x 2' 9"
Top
2-Pls. 37" x 1¼" x 7' 7½"-Side
2 Ls- 8" x 8" x 1" x 10' 9"
2 Ls- 8" x 8" x ½" x 14' 7½"
2-Pls. 6½" x ⅝" x 3' 4"
Bott.
Top Chord 23-23A
2-Web Pls. 44" x 1⅛"
8 Ls-8" x 8" x 1"
2-Pls. 6½" x 1⅛"
2-Cov. Pls. 26" x 1"
Tie Pls. 78" x ⅝"
1- " " 44" x 1"
Latt. Ls- 6" x 4" x ½"
23
2-Guss. Pls.
90" x 1" x 10' 11"
Diaph.
Web Pl. ½" thk.
Ls-4" x 4" x ½"
Diagonal 23M
Post 22-23
2-Web Pls. 42" x 1"
4 Ls- 8" x 8" x 1⅛"
Tie Pls. 55" x ⅝"
Latt. Ls-4" x 3" x ½"
Diaph.
Web Pl. ½" thk.
Ls-4" x 4" x ½"
40' 2 41/64" Point 22 to 23
M
Diagonal 22-23A
2-Web Pls. 42" x 1"
4 Ls 8" x 8" x ⅞"
Tie Pls. 55" x ⅝"
Latt. Ls 4" x 3" x ½"
Splice
2-Pls. 40" x 1⅛" x 10' 1"
4- " 15½" x 1½" x 10' 1"
4- " 15½" x 1" x 10' 1"
4- " 14⅞" x 1" x 10' 1"
4- " 11½" x 1" x 6' 1"
4- " 11" x ⅝" x 6' 1"
2-Guss. Pls. 104" x 1" x 16' 9"
42' 6"
Pt. 22 to 22A
22
Splice
2-Pls. 67¼" x 1" x 15' 10"
2- " 66" x 1" x 10' 6⅜"
2- " 16½" x ⅝" x 6' 10½"
2- " 16½" x ⅝" x 6' 5⅞"
2- " 15⅞" x ⅝" x 6' 9⅞"
2- " 15¼" x ⅝" x 6' 6⅞"
4- " 6⅞" x ½" x 3' 6⅛"
C.L. Hanger Pin
Bottom Chord 22-22A
1-Cov. Pl. 50" x 1"
2- " " 22" x ⅞"
2-Bars- 8" x 1"
Top
1-Cov. Pl. 50" x 1"
2- " " 22" x ⅞"
2-Bars- 8" x 1"
Bott.
2-Pls. 32" x 1"
2- " 32" x 1⅛"
12 Ls-8" x 8" x 1"-Top, Bott. & Cen.
8-Web Pls. 39⅛" x 2-Sides
1- " " 48" x 1¼"-Cen.

FIG. 16.

to the gusset is similar to the one described for the web members, except that the web is placed alongside of the gusset, and as all such parts of the section which are not in the plane of the gusset extend to the joint. Splice-plates are placed on each side of the web and on both sides of the flange material. Thus, again, most rivets are in double shear, and the gusset is utilized as a splice-plate.

The top chords have full bearing at the joint and, in addition, are fully spliced as regards area, section modulus, and value of the connecting rivets. This was necessitated at a few joints on account of tension stresses. However, it also conforms to the best modern practice of splicing compression members fully, and not relying on their bearing at the joint. A splice is not only subject to the direct compression stresses, but also to bending stresses, due to the function of the chord as a continuous column and to the secondary bending moments caused by the rigidity of the connections. Moreover, experience has shown that, unless the chords are very carefully assembled at the shop, no reliance can be placed on getting perfect bearing at the joints.



JOINT OF BOTTOM CHORD
FIG. 17.

Joints in Bottom Chords.—An unusual type of joint was adopted for the bottom chord. The end of one of the chord members meeting at a joint was faced to a perfect plane, and that of the other member was faced to three planes, as shown exaggerated in Fig. 17, so that, when adjoining chord members were assembled, the joint was tight only over the middle third of the depth of the chord, and each outer third formed a wedge-shaped opening $\frac{1}{8}$ in. wide. These openings gradually closed during erection as the stress increased and as the assembling bolts and drift-pins were gradually replaced by rivets.

The purpose of this joint was to concentrate the bearing pressure over the middle part of the joint, and thus avoid dangerous edge pressures. Experience with other large bridges fully justified this precaution, especially on account of the unusual width of these chords. The fact, however, that the chords were planed with an accurate, specially built, planing machine, and were assembled carefully at the shop, accounts for the accuracy with which they fitted together in the field.

The outer thirds of the joints are spliced 100%; the middle third is spliced only about 50 per cent. The aggregate area of the splicing material of the whole joint is between 70 and 80% of the effective area of the chord, but its section modulus is nearly equal to that of the main section. Counting in the bearing area of the middle third of the main section, the total resisting area at the joint is from 110 to 120% of the chord section.

Size of Rivets.—In conformity with the great size and thickness of the individual parts making up the members, and to reduce the size of the gusset-plates and splice material to a minimum, it was necessary to use rivets of the largest practicable size. Accordingly, $1\frac{1}{4}$-in. rivets were chosen for the bottom chords throughout, and for the field connections of all other truss members. These rivets have grips up to $9\frac{3}{8}$ in. All other rivets in the top chords, web members, floor-beams, track stringers, and in the heavier laterals are 1 in. in diameter. The rest are $\frac{7}{8}$ in. and less.

Lateral and Sway Bracing Between Trusses.—Each panel of the top and bottom lateral truss has two intersecting rigid diagonals, designed to resist both tension and compression (Plate XXIX). There are no transverse struts, except at the panel points near each end, where they form the top and bottom struts of the sway-frames or portals. This lateral system is more rigid and economical than one with struts and slender diagonals designed to resist tension only. All sway-frames have single intersection diagonals. All laterals and members of the sway-frames have a box section, with two or three solid webs, or, in the case of the bottom laterals in the three end panels, one solid middle diaphragm. All open sides have stiff angle latticing.

All solid web-plates are in planes parallel to the plane of the lateral truss or frame to which they belong, so that the laterals can resist effectively the secondary moments and shears due to the distortion of the lateral truss. This principle is well recognized in the design of main trusses, and is equally applicable to the design of bracing in large bridges.

Laterals are commonly proportioned to resist the stresses from wind and other lateral forces. The fact is generally overlooked that the lateral system between compression chords bears to these chords the same relation as the latticing to the different ribs making up a com-

pression member. It forms with the chords a column, which must be strong enough to resist lateral buckling as a whole, and it is evident that inadequate laterals may impair considerably the strength of the bridge as a whole.

A simple approximate rule, similar to that mentioned for the proportioning of the latticing, can be applied to the lateral system, as follows:

If a is the aggregate gross section of the compression chords of all trusses, in square inches (if the chord section varies, an average value may be taken), the transverse shearing force for which the laterals in any panel and their connections should be designed is, approximately, in pounds, $S = 400\ a$, $330\ a$, and $300\ a$, if the lateral system connects two, three, or four trusses, respectively.

It is not necessary to combine this force with the shear from the assumed wind or other lateral forces, but the laterals and their connections should be strong enough to resist either. This prevents the laterals from being made too light where the wind stress is small.

Arch Bearings.—The four arch bearings are of cast steel (Fig. 13). Each bearing has to transmit to the granite skewbacks a total reaction of 30 262 000 lb., or 700 lb. per sq. in. on a bearing area 17 ft. 6 in. square. The upper shoe, which is bolted to the end of the bottom chord, consists of two castings, each weighing 30 tons. The lower face of this shoe is perfectly plane, and bears against the convex cylindrical surface of the lower shoe. This type of bearing produces a rocking motion with little friction under the deformation of the arch.

The radius of the cylindrical surface is $r = 1\,150$ in. The maximum angular motion under live load is approximately $1°\ 30'$ up or down, which produces an eccentricity of only 2.5 in. The pressure per linear inch of line of contact is $p = \dfrac{30\,274\,000}{116} = 261\,000$ lb. Owing to the elasticity of the metal, the contact is actually over a rectangular area, the width of which is approximately $b = \sqrt{\dfrac{p\,r}{E}} = 9.5$ in., wherein E is the modulus of elasticity of the material.

The pressure per square inch increases from zero, at the edge of this area, to a maximum of $s = 0.42\sqrt{\dfrac{p\,E}{r}} = 34\,500$ lb. per sq. in.,

at the center of the bearing. The average pressure is 27 500 lb. per sq. in., which is safe.

The maximum tangential force is 3 570 000 lb., or 12% of the normal pressure, and is easily resisted by the friction. However, to prevent displacement of the upper shoe, four steel dowels, 5½ in. in diameter, are set into the lower shoe and engage holes in the upper shoe.

The lower shoe consists of eleven castings, arranged in three tiers, in which the joints between the individual castings are placed alternately parallel and at right angles to the plane of the truss, so as to insure proper distribution of the pressure. A 1-in. steel plate is placed between the lower shoe and the masonry. Sixteen anchor-bolts, 2½ in. in diameter and 10 ft. long, secure the lower shoe to the masonry.

The total weight of one complete bearing is 248 tons. For better appearance, the whole bearing is enclosed in a steel hood, which produces the effect of a massive pedestal. No mortar or other bed was used between the base plate and the masonry. The skewbacks were carefully dressed to a perfect plane on which dry cement powder was evenly distributed, before the base plate was placed, so as to fill out any unevenness. The holes for the anchor-bolts were drilled to a template after the exact location of the bearings had been fixed.

The failure of the suspended span of the Quebec Bridge, during its erection in 1916, which is ascribed to the breaking of one of the cast-steel bearings, has brought to the foreground the question as to whether cast steel is a suitable material for bridge construction.

Cast steel for bearings has the advantage that it can be built into forms and thicknesses for which riveted work is impracticable. Large steel castings, however, are difficult to make. Experience has shown that unless great care is taken in casting the metal, internal defects may develop, which are difficult to detect. Further, unless properly annealed, as soon as the cores are removed, or after any subsequent local heating, internal stresses will remain in the casting, and may be large enough to cause breakage under slight shock. This applies particularly to complicated castings, or castings in which the metal varies considerably in thickness and, therefore, does not cool uniformly.

Simplicity and uniform thickness, therefore, should be the aim in designing the castings. Provision should also be made to insure quick removal of the cores; otherwise, their resistance to the free shrinkage

of the metal, while it cools, may cause serious stresses in the latter and permanent defects which may not be removed by the subsequent annealing. Bearings which have to distribute a concentrated load over a certain area should have a height of at least one-half the width of the bearing area and be safely proportioned for the bending and shearing stresses. If these precautions are taken in the design and manufacture, and scrupulous inspection is exercised, there need not be any apprehension concerning the use of cast steel for bearings.

Stringers and Floor-Beams.—Typical details of the floor system are shown in Plates XXX and XXXI. The railroad stringers are 6 ft. deep, and of the ordinary make-up. They are framed into the floor-beams. The connection was designed to allow the stringers to be swung into place after the floor-beams were erected. The stringers are connected in pairs by top and bottom lateral bracing and two sway-frames in each panel.

The floor-beams are heavy box-girders, with two webs, 8½ ft. deep. The webs are joined by top and bottom cover-plates, 42 in. wide, and by vertical diaphragms in the lines of the stringers and horizontal diaphragms opposite the floor-lateral connections. On the suspended portion of the floor the floor-beams are continuous between railing girders and connected to the floor suspenders by 16-in. pins. At the four end panel points, the floor-beams are framed into the truss verticals, and separate cantilever brackets are provided on the outside. One of the floor-beams weighs 86 tons.

Floor-Lateral Truss and Braking Girders.—The floor-lateral truss is in the plane of the bottom flanges of the railroad stringers, the diagonals being riveted to the stringers at intersection points and also to the floor-beams. The diagonals take tension and compression. Each diagonal consists of a horizontal web-plate with two or four angles riveted to its lower side (Plate XXXI). On account of the great unsupported length between the outer stringer and the wind chord, the diagonals are stiffened in that portion by a latticed strut parallel with and 2 ft. above the diagonal, and connected to the latter by lattice angles.

At the expansion joint of the floor, the two diagonals, on the sliding side, are connected to a horizontal gusset-plate which slides longitudinally, but transmits the lateral reaction to a bracket attached to the center of the floor-beam (Plate XXXI). The chords of the

PLATE XXXI.
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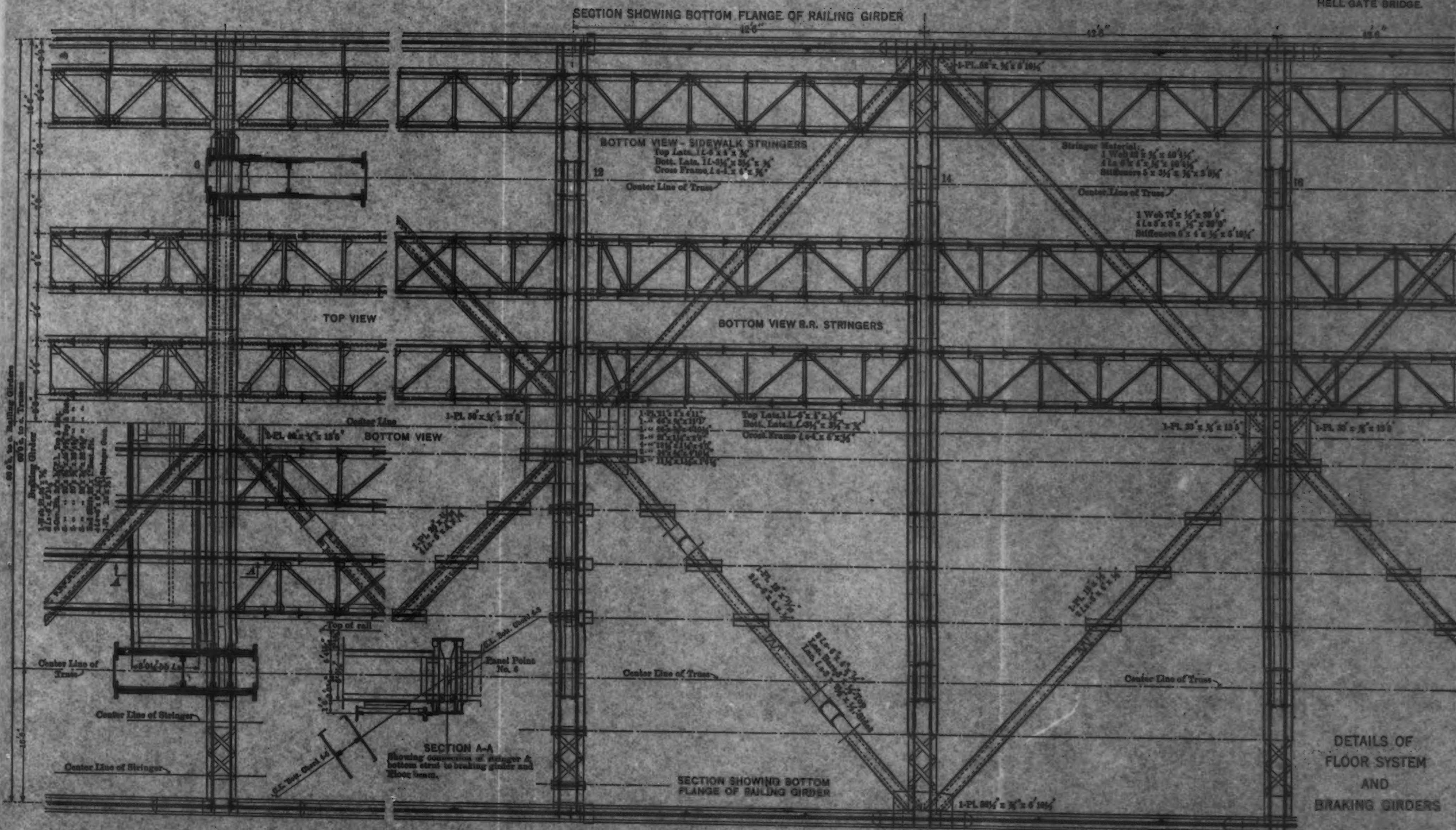
HELL GATE BRIDGE.



DETAILS OF
FLOOR SYSTEM
AND
BRAKING GIRDERS

floor-lateral truss which form the bottom chords of the railing girders have the shape of an inverted T, the vertical web-plate forming the gusset for the web members of the railing girder. To reduce the unsupported length of this chord between floor-beams, the railing girder is connected with the highway stringers by intermediate brackets, one opposite each stringer cross-frame (Plate XXX).

The two braking girders at the intersections of the floor with the trusses are horizontal, single-web plate girders, 8 ft. wide, and placed immediately below the floor laterals (Plate XXXI). These girders are rigidly framed into the bottom chords of the arch trusses (Fig. 14) to which they deliver their reactions. Each railroad stringer is connected to the braking girder by a vertical diaphragm.

Steel Weights.—The weight of the steel superstructure of the Hell Gate Bridge is made up as follows:

Stringers	4 010 000	lb.
Floor-beams	3 870 000	"
Railing girders with wind chords	635 000	"
Floor laterals and braking girders	575 000	"
Total floor system	9 090 000	lb.
Arch trusses	21 640 000	"
Floor suspenders	1 380 000	"
Arch bracing	3 670 000	"
Bearings	1 990 000	"
Total steel weight	37 770 000	lb.

or an average of 37 000 lb. per lin. ft. of bridge. This does not include 470 tons of I-beams in the concrete floor-slabs.

7.—Camber and Deformation of Arch Trusses.

For a large bridge of unusual design, an investigation of the elastic behavior of its trusses under load is essential for a proper judgment of its merits. The elastic line gives at once an indication of the comparative rigidity of a bridge, and as to whether and where large secondary stresses may occur. Further, the calculation of the greatest deflection is necessary for determining the proper camber, in order to satisfy clearance conditions under the bridge, and to establish the desired grade of the floor.

Camber.—The arch trusses of the Hell Gate Bridge were cambered for dead load only by increasing or decreasing the "geometric length" of each member, including the floor suspenders, by an amount equal—but opposite—to its change in length from its dead-load stress, so that, under full dead load, the trusses would assume their true "geometric form", for which the stresses are calculated. This is the proper method of cambering large bridges. There was no need for cambering the trusses for live load, as the deflections have only a negligible influence on the direct stresses in the truss members. The only object was to prevent the top of rail from sagging below a horizontal line under full live load and extreme low temperature, and this was accomplished by establishing an initial vertical curve for the top of rail.

Deflections.—Plate XXXII shows the assumed initial vertical curve, and gives a summary of the vertical deflections for the points along the top of rail. In the calculation for the deflections, the "gross area" of all members was assumed, and the influence of the details, and of the rigidity of the connections, was neglected. The modulus of elasticity was assumed to be 30 000 000 lb. per sq. in., and the linear temperature expansion as 0.0000065. The effect of the elastic deformation of the web members on the deflections of an arch of this type is very considerable, especially for partial load, and cannot be neglected.

Dead-Load Deflections.—The maximum vertical deflection of the floor, due to the total dead load (average 51 000 lb. per ft. of bridge), from the theoretical cambered position, is 8.35 in. at Panel Point 22 (Wards Island side) and 7.13 in. at Panel Point 22 (Long Island side). This difference for the two sides, and the kink at Panel Point 22, Wards Island side, in the cambered position, is due to the fact that, in its cambered position, the truss forms an unsymmetrical, three-hinged arch, and was erected as such, the hinge being at Panel Point 22, Wards Island side, whereas, in the final position, the arch was to assume its true geometric form, which is symmetrical about the center.

Live-Load Deflections.—Plate XXXII (*b*) shows the elastic lines of the floor for a live load of 12 000 lb. per ft. of truss covering one half span and the whole span, respectively, and also the lines connecting the extreme positions of the panel points under most unfavorable positions of the load. The greatest downward deflection is 5.19 in., or $\frac{1}{2\,300}$ of the span length, and occurs at about the quarter point of the

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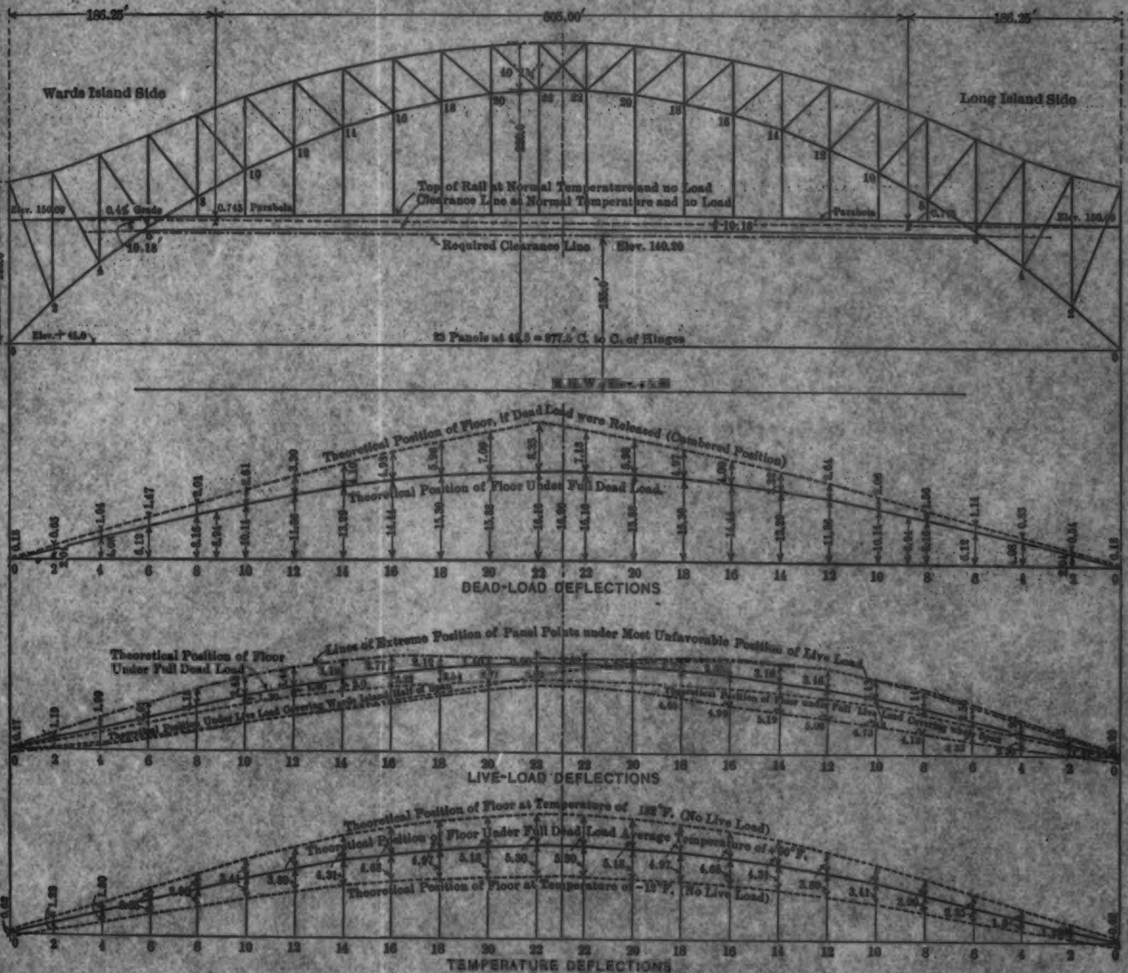
DEFLECTIONS OF ARCH TRUSSES

Note:- All Deflections in these Diagrams are those of Floor Level, and are given in Inches
+ Denotes Downward Deflections, — Upward Deflections.



span under a load covering about one-half of the span. The quarter point on the other half of the span rises 3.46 in. under the same loading condition. The greatest deflection at the center is 3.82 in., or $\frac{1}{3\,100}$ of the span length, and occurs under a load covering approximately the middle half of the span. The maximum live-load deflection at the center of a cantilever bridge of the same span and loading would be about 8 in., and that of a simple span about 5 in.

The elastic lines are smooth curves, or rather polygons, free from local kinks, such as occur at the hinges of cantilevers or three-hinged arches. They are also free from sharp corners, such as are produced over the intermediate supports of cantilevers, or at every panel point of trusses with subdivided panels. From this it may be concluded that the secondary stresses in the arch trusses are comparatively small, which fact is substantiated by the calculations of these stresses.

Temperature Deflections.—A change in temperature from the normal (60° Fahr.) produces an elastic line of approximately parabolic shape with greatest ordinate at the center of the span (Plate XXXII (c)). The latter rises or falls, respectively, by 0.74 in. for each degree of rise or fall in temperature, the total deflection for a change of 72° Fahr. being ± 5.3 in., or $\frac{1}{2\,250}$ of the span length. This additional deflection from temperature change is not objectionable, as it has no bearing on the rigidity of the bridge. The temperature deformation is the result of the change in length of each member, first, due to unconstrained expansion or contraction, and, second, due to the elongation or compression from temperature stress. The former effect raises or lowers each point in proportion to its height above the bearings.

8.—MATERIAL.

Quality of Steel.—All material in the steel superstructure of the Hell Gate Bridge and Approaches is rolled, forged, or cast steel, made by the open-hearth process.

The following four grades of steel were used:

a.—Hard steel for all rolled parts and pins of the Hell Gate Arch Bridge;

b.—Structural steel for all rolled parts and pins of the Approaches;

c.—Rivet steel for all rivets;

d.—Cast steel for all castings for bearings, etc.

These different grades had to conform to the chemical and physical requirements given in Table 1.

TABLE 1.—CHEMICAL AND PHYSICAL REQUIREMENTS FOR STEEL.

		Hard steel.	Structural steel.	Rivet steel.	Cast steel.
Phosphorus, max.	basic	0.04	0.04	0.04	0.05
	acid	0.06	0.06	0.04	0.08
Sulphur, max.		0.05	0.05	0.04	0.05
Ultimate tensile strength, in pounds per square inch.	max.	76 000	70 000	58 000	
	desired	71 000	66 000		
	min.	66 000	62 000	50 000	65 000
Yield point, min.		38 000	35 000	28 000	33 000
Elongation, min.	in 2 in. for cast steel. / in 8 in. for other steel.	*	22%	28%	20%
Character of fracture		Silky.	Silky.	Fine, silky.	Silky or fine granular.
Cold bend without fracture		*	180° around pin of thickness of test piece.	180° flat.	90° around pin of thickness of test piece.

* Minimum elongation for "hard steel": 1 400 000 divided by ultimate strength for thicknesses up to and including ¾ in.; 1% less for each additional ⅛ in. in thickness, with a limit of 16% for thicknesses up to and including 2 in. and 15% for thicknesses greater than 2 in.

Cold-bend test for "hard steel": 180° around a pin of double the thickness of the test piece for material up to and including ¾ in., 180° around a pin three times that thickness for material of greater thickness than ¾ in.

The chemical analysis was made from test ingots during the casting of the melt, and included the determination of carbon, manganese, and silicon, in addition to phosphorus and sulphur. The carbon ranged between 0.27 and 0.34% in the hard steel and between 0.23 and 0.28% in the structural steel, and manganese from 0.52 to 0.64 and 0.36 to 0.61%, respectively. Special stress was laid on sufficient discard being made from the ingot to insure sound material free from piping and excessive segregation.

The physical tests were made on standard test specimens cut from the rolled, forged, or cast piece, in the latter two cases after annealing. The specifications required that at least two tensile and two bending tests be made from each heat of 25 tons or less, at least three tests from each heat up to 40 tons, and four tests for heats exceeding 40 tons. If the material rolled from the same heat varied in thickness ⅜ in., or more, the foregoing number of tests were to be made from the thickest as well as from the thinnest pieces. In all about 7 000 tests, covering

accepted heats only, were made, or an average of eight tests per 100 tons.

Full-Sized Eye-Bar Tests.—About 1 900 tons of structural steel eye-bars, 16 in. wide and from $1\frac{7}{8}$ to $2\frac{3}{8}$ in. thick, with forged heads $37\frac{1}{2}$ in. in diameter, and 16-in. pin-holes were used in the Little Hell Gate Bridge. Twenty-one full-sized bars were tested, after annealing, and had to show the following results: Ultimate strength: 56 000 to 68 000 lb. per sq. in.; elastic limit: minimum, 33 000, and maximum, 38 000 lb. per sq. in.; minimum elongation in 10 ft.: 12 per cent.

Table 2 gives a summary of the full-sized tests, together with the corresponding unannealed specimen tests.

TABLE 2.—RESULTS OF TWENTY-ONE FULL-SIZED EYE-BAR TESTS.

	Annealed Full-Sized Bars.			Unannealed Specimen.			Difference between average full size and specimen.
	Min.	Average.	Max.	Min.	Average.	Max.	
Ultimate strength, in pounds per square inch	58 500	61 800	64 700	64 300	67 800	72 600	6 000
Elastic limit, in pounds per square inch	32 900	34 600	36 200	35 700	38 800	41 000	4 200
Elongation: percentage in 10 in.	17.8	22.8	29.8	....	....	....	....
Elongation: percentage in 12 in.	31.6	39.3	49.8	in 8 in., 21.2	26.4	30.0	....
Reduction: percentage	29.1	36.0	43.2	30.0	36.9	43.0	....

All specimens broke in the body of the bar.

Selection of High-Carbon Steel for Arch Bridge.—A high-grade carbon steel was adopted for the Hell Gate Bridge, principally because it permitted higher unit stresses, which resulted in a considerable saving in steel. The consequent saving in cost was offset only to a small extent by the slightly greater unit price of hard steel over ordinary structural steel. This slightly greater price is not due so much to greater cost of manufacture at the mill and shop as to the fact that, for any special steel, the quantity to be scrapped, on account of not fulfilling the requirements, is greater than for ordinary material. The contractor was given the option of furnishing, at the same price, either hard or structural steel for the floor system and suspenders, which parts have been designed with the unit stresses allowed for structural steel. He preferred, however, to use the same material, that is, hard steel, for the whole bridge. Further, with ordinary steel, the sections

of truss members, gusset-plates, splices, and size of rivets would have become much larger and, in some cases, excessive. Even with the hard steel as used the dimensions of some of these details are up to the practicable limits.

The hard steel showed an elongation of generally more than 20%—in the average about 25%—and an average reduction of 45%, and no difficulties were experienced in complying with the bending test. This proves that the material is tough and ductile. It required specially hard tools for drilling and reaming, and these operations were somewhat slower than with ordinary carbon steel, but otherwise no serious difficulties were experienced in the fabrication.

Adaptability of Alloy Steel.—The use of nickel steel for the arch trusses was also taken into consideration, but it was found that, with permissible unit stresses 50% higher than for structural steel, there would have been no saving in cost. The unit price for nickel steel obtaining at that time was about $40 per ton higher than for carbon steel. Moreover, unless nickel steel, or any other alloy steel, would have resulted in an appreciable saving, or other important advantages, carbon steel would still have been given the preference. Carbon-steel trusses, on account of their greater weight or inertia and smaller elastic deformations, are less subject to vibrations from live load, and, consequently, insure greater stiffness and permanency. Moreover, as the proportion of their dead weight to live load is greater, they will sustain safely a comparatively greater future increase in live load than alloy steel.

9.—Loads and Unit Stresses.

The safety and useful life of a bridge depend essentially on the proper assumption and co-ordination of the various loads and permissible unit stresses. In the design of bridges of ordinary span and capacity, the assumption of loads and unit stresses is a matter of well-established routine, whereas, for a large bridge, for which there are few or no precedents, it is a complex problem, which must be solved largely by judgment. The more accurate and complete the load assumptions, the higher can be the permissible unit stresses. It is safer and more in conformity with true economy to make sure that under the most unfavorable, but possible, conditions and combinations of loading the elastic limit of the material or a comparatively high proportion thereof will not be exceeded, than to trust to ordinary

loading conditions and low unit stresses and allow a large, but uncertain, margin of safety. This principle has been followed in the design of the Hell Gate Bridge. All possible forces have been taken into consideration, namely, dead load, live load, vertical impact, lateral force or impact, longitudinal force from traction and braking of trains, wind pressure, and forces due to change in temperature, and comparatively high unit stresses (from five-eighths to three-fourths of the minimum elastic limit) have been allowed for the combination of these forces.

Dead Load.—Before determining on the final sections of the truss members, the dead load was carefully calculated and checked. From the preliminary designs and detailed drawings the panel concentrations were ascertained very closely. After the work had been let a recalculation was made from drawings worked out by the contractor sufficiently in detail to allow the ordering of material therefrom.

The dead load, in pounds per linear foot of bridge, is made up as follows:

Tracks and ballast....................	4 900 lb.	
Steel concrete and timber flooring......	8 100 "	
Conduits, cables, wires, etc...........	1 000 "	
Total tracks and flooring, etc.................		14 000 lb.
Steelwork, average..............................		37 000 "
Total average dead load per foot of bridge....		51 000 lb.

The dead load assumed for the final calculation is 51 900 lb., average, and varies from 45 000 lb. at the center to 62 000 lb. at the ends. The excess over the actual dead load is due to the excess of the assumed over the actual weight of the conduits. For the weight of steelwork (exclusive of steel in floor slabs), the stresses have been calculated by assuming the arch trusses to be three-hinged, under which condition they were erected (middle hinge at bottom chord, Point 22, Wards Island side, Plate XXVIII). The stresses for the remainder of the dead load, as well as for all other forces, have been calculated for the final two-hinged condition.

Live Load.—The arch bridge and approaches are designed for the following live load:

1.—All bridges and viaducts of the approaches, and the floor system and floor suspenders of the arch bridge, for Cooper's E-60 on each of the four tracks, or an alternative three-axle load of 70 000 lb. on each axle wherever this causes greater stresses (Fig. 18).

2.—The arch trusses for a uniform load of 6 000 lb. per ft. of track, or 24 000 lb. per ft. of bridge, placed in the most unfavorable position in either a single stretch or in two separated stretches, when the latter condition gives a greater stress.

The assumption of a uniform train load for the arch trusses, instead of the engine concentrations, was justified in view of the highly improbable, and almost impracticable, condition of maximum engine and car loads, in the most unfavorable position simultaneously on all four tracks. This hypothetical condition would have required addi-

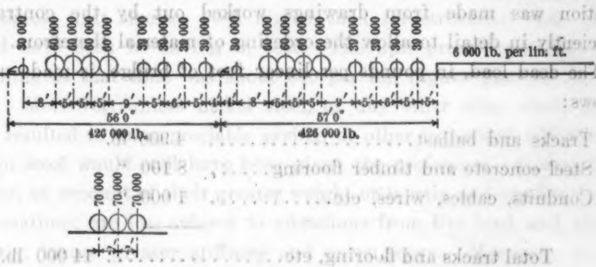


DIAGRAM OF ASSUMED LIVE LOAD

FIG. 18.

tions up to about 5% to the sections of some of the truss members. An E-75 loading on each track in the most unfavorable position, but without impact, would have increased the sections of a few of the heaviest bottom chord members by not more than 3½%, the sections of all the other members being ample. Such a heavy load, if at all possible, with present limits of gauge and clearance, represents probably the heaviest future freight trains, and as these will move comparatively slowly, the impact effect of which on the truss members will be negligible, it is considered, therefore, that the bridge will carry safely any possible future moving load.

Impact.—The impact stresses, or vertical dynamic effects of the locomotives and cars, have been determined according to Lindenthal's formula, as published and fully explained in an article* by its author.

* *Engineering News*, August 1st, 1912.

Since the appearance of that article, the formula has been modified slightly so as to be applicable automatically to bridges having two or more tracks.

The formula in its final form, applied to E-60 loading, is:

$$I = \frac{L^2}{D+L} \times \frac{1\,200 + \frac{a}{n}}{600 + 4a}$$

wherein, D = stress from dead load, in pounds,

L = stress from live load, in pounds,

a = length of train behind locomotive tender for position of maximum stress, in feet,

n = number of tracks loaded for maximum stress.

The modification of the formula consists of the introduction of the value, n, and is based on the assumption of full impact from the locomotives on all tracks, but impact from the cars on one track only. The principal object sought in deriving this formula was that it should be applicable equally to all kinds of bridges, of the shortest as well as the longest spans, to bridges with open or solid ballasted floors, and to steel or concrete bridges; whereas most of the existing formulas have only limited usefulness, particularly the widely adopted formula of the American Railway Engineering Association. The latter formula gives excessive results for long spans and heavy ballasted bridges, and insufficient values for very short stringers, rails, ties, etc., on which the dynamic effect of the driving wheels is unquestionably more than 100 per cent. Lindenthal's formula, in combination with the comparatively high permissible unit stresses, secures economical and well-proportioned structures. Seemingly, it is complicated, but, by tabulation or graphical diagram, its application is very simple.

Lateral Force from Live Load.—The lateral force or lateral impact, due to the swaying motion of fast-moving trains on tangents, or the centrifugal force on curves up to 2°, has been assumed at 600 lb. per lin. ft. of single track. For curves sharper than 2°, 300 lb. were added for each additional degree up to 6 degrees. These forces were increased by 50% for each additional track. This gave, for a four-track bridge on tangent, a total lateral force of 1 500 lb. per ft.

The lateral force occurs simultaneously with the live load, and may act at the same time as the wind pressure, and it is proper, therefore, to provide for it in the proportioning of the laterals as well as the

main truss members. The foregoing value of the lateral force, for a four-track structure, is probably higher than any which may ever actually occur, but it tends to secure a strong and rigid lateral system along the floor. As far as the writer knows, this is the first attempt to make adequate provision for the lateral force from trains on tangents, and to separate it clearly from the wind pressure. The American Railway Engineering Association Specifications of 1914, in this case, would call for a total combined lateral and wind force of only 800 lb. per lin. ft. of floor.

Wind Pressure.—Wind pressure was assumed as a moving load of 500 lb. per lin. ft. of bridge at track level, irrespective of the number of tracks, plus a static load of 30 lb. per sq. ft. on all such vertical surfaces of the unloaded bridge as are exposed at an angle of between 20° above and 20° below the horizontal, or at an angle of 45° from the axis of the bridge. This resulted, for the Hell Gate Arch, in an average wind pressure of 600 lb. per lin. ft. of horizontal projection of top chord, 1 000 lb. per lin. ft. of horizontal projection of bottom chord, and 1 500 lb. per lin. ft. of floor (inclusive of the 500 lb. moving wind pressure), or a total of 3 100 lb. per lin. ft. of bridge. Adding to this the lateral force of 1 500 lb., it is seen that, with a total force of 4 600 lb. per ft. of bridge, ample lateral strength, rigidity, and stability are secured.

Longitudinal Force from Traction and Braking.—The longitudinal force, acting along the rail, from traction and braking has been assumed either at 15 000 lb. for each of the eight driving axles of the two locomotives (25% of the load on each driving axle), or at 1 000 lb. per lin. ft. of train (approximately 15% of the average weight of the train), whichever gave the greater results. For the four-track bridges this force was assumed as acting on two tracks only, in view of the practical impossibility of its acting simultaneously on all four tracks in the same direction.

Recent tests with improved air-brake equipment, yielding a considerably increased brake power, have led to suggestions that structures be designed for a higher braking force than has been the practice heretofore. The effect of the more powerful brakes, that is, the greater force exerted by the brakes to the wheels, is to reduce the time in which a stop can be made without causing dangerous slipping of the wheels on the rails; in other words, the braking force between wheel

and rail attains its maximum value in a shorter time. Its amount, which depends solely on the friction between rail and wheel, is not increased. For reasons of safety, the action of the brakes will never be so sudden that the braking force at the rail acts with impact, that is, the braking force can always be regarded as a static force which increases gradually to its maximum value.

The friction between wheels and rails has been determined variously at from 15 to 30% of the total vertical load on the wheels to which brakes are applied, the percentage depending on the smoothness of the rail surface under varying climatic conditions. As the greatest possible braking force of the two heaviest trains will rarely, if ever, be applied to the bridge, the assumed longitudinal force appears to be ample, no matter how efficient the brake equipment may be made in the future.

Temperature Stresses.—The stresses from temperature have been determined for a variation of ± 72° Fahr. from the normal temperature of 60° Fahr. The temperature stresses are greatest in the center panels of the top chord of the arch trusses, where they attain values of 4 000 lb. per sq. in., or about 40% of the live-load stresses.

Total Stresses.—All members were proportioned for the following combination of stresses:

1.—For members which carry dead and live load, the "total stress" was obtained by adding the stresses from dead load (D), live load (L), impact (I), lateral force (Lat.), and the so-called "excess stress" (Exc.). This excess stress is the sum of the stresses from wind pressure (W), braking force (B), and temperature (T), less 20% of the sum ($D + L + I +$ Lat.). This is equivalent to allowing up to 25% higher unit stresses for the sum ($D + L + I +$ Lat. $+ W + B + T$) than for the previously mentioned "total stress", the percentage decreasing with increasing value of ($W + B + T$).

2.—For members which carry no dead and live load, the "total stress" was made equal to ($W + B + T +$ Lat.).

The "total stress," divided by the permissible unit stress, gave the minimum required area.

Permissible Unit Stresses.—The permissible unit stresses assumed are given in Table 3.

The stresses in Table 3 are higher than those allowed by most specifications, notably those of the American Railway Engineering

TABLE 3.—PERMISSIBLE UNIT STRESSES ASSUMED.

		For Trusses and Bracing of Hell Gate Bridge. (Hard steel). In pounds per square inch.	For Approach Spans and Floor System and Suspenders of Hell Gate Bridge. (Structural steel). In pounds per square inch.
Axial tension, net section		24 000	20 000
Bending on extreme fiber of beams, girders, and steel castings, net section		......	20 000
Axial compression, net section:			
(a) Closed section, or section with two diaphragms, or one diaphragm and two planes of latticing	$\frac{l}{r} = 20$	24 000	20 000
	40	23 000	19 000
	60	22 000	17 000
	80	20 000	15 000
	100	18 000	14 000
	120	15 000	12 000
(b) Half-open section, with one cover and one latticing, or with one diaphragm without latticing	$\frac{l}{r} = 20$	23 000	20 000
	40	22 000	18 000
	60	20 000	16 000
	80	18 000	14 000
	100	16 000	13 000
	120	14 000	11 000
(c) Open section, with two or more planes of latticing	$\frac{l}{r} = 20$	22 000	19 000
	40	20 000	17 000
	60	18 000	15 000
	80	16 000	13 000
	100	14 000	12 000
	120	12 000	10 000
Shearing stress:			
On plate girders, net section		.....	15 000
Shop rivets and pins		15 000	
Field rivets and turned bolts		12 000	
Bearing stress:			
Pins		24 000	20 000
Shop rivets		30 000	
Field rivets and turned bolts		24 000	
Pressure on:			
Expansion rollers per linear inch		Diameter of roller, in inches, × 1 000	
Granite masonry		800	800
Concrete masonry		600	800

Association. In combination with Lindenthal's impact formula, however, they result in heavier bridges for spans up to about 250 ft. and lighter ones for longer spans. The assumption of the permissible compression stress per unit of net area, that is, with rivet holes deducted, is unusual. Most specifications base the compression stress on the gross section, the assumption being generally made that the rivet shanks replace the compression value of the metal cut out for the holes. This assumption is proper, provided the rivet fills

the hole perfectly, and its metal has the same elastic properties as that of the member. To a certain extent, also, the friction of the rivet heads on the member makes up for loss in compression strength due to the holes.

These conditions, however, are not always fulfilled, and, moreover, compression members are subject to shearing stresses and, near the point of failure, possibly even to tension stresses, both of which cannot be resisted by the rivets, or only to a limited extent. It is evident, therefore, that rivet holes, even after riveting, decrease the strength of a compression member. This decrease may not be appreciable in ordinary compression members in which the rivets have short grips, generally fill the holes well, and develop a comparatively high friction under their head. It is to be assumed, however, that in heavier members the decrease in strength, due to the holes, is greater on account of the longer grip of the rivets and the probability of less perfect rivets.

The extent to which compression members are weakened by the holes can only be determined by a systematic series of comparative tests, embracing both light and heavy sections. In the absence of sufficient experimental data, it is advisable to remain on the safe side.

Another unusual feature of these specifications is the diversity of permissible stresses for different types of compression members, greater stresses being allowed for members having solid diaphragms or cover-plates than for those having flanges which are connected by latticing. The usual compression formulas do not discriminate in this respect, the only provision, in most specifications, is that the portion of the flange between connections of the latticing shall be as strong as the member as a whole. This is usually interpreted to mean that the ratio, $\frac{l}{r}$, of that portion of the flange shall not be less than for the member as a whole, and implies that the strength of the member is the same for any value, $\frac{l}{r}$, of the flange, within the value, $\frac{l}{r}$, of the member as a whole. This assumption is fallacious. The flange of a member, subject to buckling, should have an unreduced compressive value, the same as the flange of a beam subject to bending. If it has not, that is, if the flange itself has a reduced buckling strength, then the strength of the compression member as a whole is doubly reduced.

There are other weaknesses of integral parts, such as insufficient thickness of webs, cover-plates, outstanding flanges of angles, excessive rivet pitch, etc., all of which tend to reduce the strength of a compression member as a whole. It is evidently impossible to cover all these influences by cut-and-dried rules or formulas, particularly in view of the lack of sufficient comparative experimental data, but it is obvious that some distinction should be made in the permissible unit stresses for different types of sections, types of latticing, etc., and not merely for different values of the ratio, $\frac{l}{r}$.

Secondary Stresses.—Care was taken in designing and detailing all the bridges to avoid large secondary stresses, and, in a few cases, where this was not possible, additions to the sections were made.

In the Hell Gate Arch the maximum calculated secondary stresses which occur simultaneously with the maximum primary stresses are as follows, in pounds per square inch:

	Dead load.	Live load.	Combined dead and live load.
Bottom chord........	$\pm$ 1 300	$\pm$ 1 300	$\pm$ 2 100
Top chord..........	+ 2 100	$\pm$ 3 100	+ 5 200
	— 2 500		— 5 600
Diagonals	$\pm$ 5 000	$\pm$ 2 800	$\pm$ 6 300
Verticals	$\pm$ 2 200	$\pm$ 4 400	$\pm$ 4 800

These, being extreme fiber stresses, can safely be assumed to be covered by the margin of safety of the primary or axial stresses, especially as the greatest secondary stresses occur in members which have considerable excess of section. Moreover, stress measurements made during and after erection indicate that the actual secondary stresses are below the calculated ones.

No attempt, therefore, was made in the fabrication and erection of the trusses to eliminate or even reduce the secondary stresses. In fact, this would have been a very difficult problem on account of the rigidity of the members, particularly of the bottom chord. This question is discussed more fully under "Workmanship and Fabrication."

Erection Stresses.—The greatest stresses in the arch trusses during erection were: + 18 600 and — 16 600 lb. per sq. in. $\left(\frac{l}{r} = 46\right)$ from dead load and erection traveler without wind, and + 20 400 and — 19 700

lb. per sq. in. $\left(\frac{l}{r} = 46\right)$ from the dead load, traveler, and an assumed wind pressure of 30 lb. per sq. ft. of exposed surface.

These stresses were well within the safe limits allowed for the total stresses in the completed bridge. Only in the end panel of the top chord was the erection stress greater than the total stress in the permanent bridge, but these members required a certain minimum section which was sufficient for the erection stress. No extra metal was required, therefore, in the arch trusses for erection purposes. The maximum values allowed for the erection stresses without wind in the temporary back-stays (structural steel) were the same as those allowed for the total stresses in the finished structures (20 000 lb. per sq. in. tension) and 25% more with a wind pressure of 30 lb. per sq. ft. of exposed surface (25 000 lb. per sq. ft. tension).

10.—Workmanship and Fabrication.

The specifications were prepared by the Consulting Engineer with a view to securing the best class of workmanship which current practice and possible improvement therein would admit. The steelwork for the arch bridge was manufactured at the Ambridge Plant of the American Bridge Company, whose shop management gave careful study to new shop methods and the use of more efficient tools and machinery, as necessitated for the unprecedented work.

Punching, Reaming, and Drilling.—The punching, reaming, and drilling of rivet holes was governed by the following clauses in the specifications:

"Steel up to and including a thickness of $\frac{1}{2}$ in. may be punched without subsequent reaming, unless the same shall be necessary to insure smooth holes of the assembled parts to be riveted together.

"Steel up to and including $\frac{5}{8}$ in. in thickness shall be punched with holes $\frac{1}{8}$ in. smaller and then reamed after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivets shown on the drawings.

"Structural steel up to and including $\frac{3}{4}$ in. in thickness may be punched and reamed, provided that in no instance shall the punched rivet hole be more than $\frac{11}{16}$ in. in diameter in steel $\frac{3}{4}$ in. thick. This applies to any diameter of rivet, whether $\frac{7}{8}$ in. or larger. After assembling the holes shall be reamed out for the removal of the bruised metal to a diameter $\frac{1}{16}$ in. larger than the nominal size of the rivet. A misfit or overlapping of the punched holes in the assembled pieces greater than $\frac{1}{16}$ in. will not be allowed, and when the Engineer is satisfied, after reasonable trial, that the punching is not within that limit

of accuracy then punching must be discontinued and the holes drilled as per paragraph 55.

"Rivet holes of more than $\frac{5}{8}$ in. in thickness shall be either drilled from the solid $\frac{1}{16}$ in. smaller and then reamed after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivets shown on the drawings, or the rivet holes shall be drilled from the solid after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivet.

"Rivet holes in members composed of material of two or more thicknesses, any one of which is greater than $\frac{5}{8}$ in., shall be punched or drilled, respectively, as specified in paragraphs 54 and 55, and after assembling reamed to proper size, or the rivet holes shall be drilled from the solid after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivet.

"All reaming or drilling of holes in assembled parts shall be done with the various pieces bolted together in their respective relative and exact positions. After reaming, every hole shall be entirely smooth, showing that the reaming tool has everywhere touched the metal.

"All drilling and reaming of holes shall be done dry with self-hardening tool steel.

"After the drilling or reaming is completed, a special reamer shall be run over both edges of every rivet hole to remove the sharp edges and burrs and make a fillet $\frac{1}{16}$ in. deep in each rivet head. When necessary the work shall be taken apart and any shavings between pieces carefully removed."

On account of the thick material composing the truss members of the arch bridge, nearly all the holes in these members had to be drilled. The general procedure was as follows: After the holes had been laid out with wooden templates and punch-marked, from 10 to 15% of all of them were drilled from solid to a diameter of $\frac{11}{16}$ in. The parts were then assembled and bolted up with $\frac{5}{8}$-in. tack-bolts. All the other holes were then drilled from the solid to full size, except those for the field rivets, which were also drilled to a diameter of $\frac{11}{16}$ in. The holes of the $\frac{5}{8}$-in. tack-bolts were then reamed to full size, the bolts being gradually replaced by larger ones, and finally the sharp edges of the holes were reamed off. The member was then riveted.

For the bottom chord members of the arch, the two webs with their flanges had first to be assembled and riveted separately and then joined and riveted to the middle diaphragm and finally the top and bottom cover-plates were added. All reaming and drilling of

the hard steel was done with tools of special high-speed steel. Lubricating with oil was not permitted, but it was found necessary to use occasionally a few drops of soap water to reduce the friction, particularly in drilling through thick material.

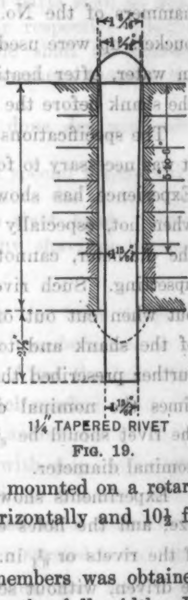
Rivets and Riveting.—There are, in the whole arch bridge, about 840 000 shop rivets and 334 000 field rivets, of which 400 000 are 1¼ in. in diameter. Most of the 1- and 1¼-in. shop rivets were driven with hydraulic riveting machines having a pressure capacity of 100 tons (100 lb. per sq. in.). The output of one of these machines is about 3 500 rivets per day. For all heavy field rivets and such shop rivets as could not be driven with pressure machines, pneumatic riveting hammers of the No. 90 Boyer type with 9-in. stroke and pneumatic buckers-up were used. The points of the long rivets had to be dipped in water, after heating, so as to insure more complete upsetting of the shank before the head was formed.

The specifications required the rivets to be of such a diameter that it was necessary to force them into the holes with a hammer when hot. Experience has shown that rivets which drop easily into the holes when hot, especially those of long grip exceeding about three times the diameter, cannot be relied on to fill the holes completely after upsetting. Such rivets may seem tight when tested with a hammer, but when cut out often show incomplete upsetting near the middle of the shank and toward the shop-formed head. The specifications further prescribed that rivets with a grip equal to, or exceeding, four times the nominal diameter should be tapered so that the base of the rivet should be $\frac{3}{64}$ in. larger and the point $\frac{1}{64}$ in. smaller than the nominal diameter.

Experiments showed that such rivets, when smooth and of perfect size, and the holes exactly $\frac{1}{16}$ in. larger than the nominal diameter of the rivets or $\frac{1}{64}$ in. larger than the actual diameter at the base, can be driven, without scraping the hot metal of the shank and forming a film under the rivet head, and fill the holes more perfectly than ordinary cylindrical rivets. It was found, however, that allowance had to be made for the irregularity in diameter of rivets and holes due to the rapid wear of the rivet dies and the drills. After extended experiments, the following size and shape was finally adopted for the 1¼-in. rivets (Fig. 19): Minimum diameter under the head, $1\frac{9}{32}$ in., tapered

down to $1\frac{15}{64}$ in. in a distance of from 5 to 6 in. from the head, depending on the length of the rivets, the rest of the shank being cylindrical.

The rivets were required to be perfectly round, and free from loose scale and projecting fins, and before they were entered into the holes, the scale, formed in heating, had to be carefully scraped off. To produce perfectly round and smooth rivets, it was found necessary to deviate from the ordinary practice of upsetting the rivet between dies by a single stroke of the machine and cutting it off at the same time from the feeding rod. Pieces of the proper length required for the various rivets were cut cold from the rivet rods and, after heating, were placed individually into the upsetting machine. Each rivet was given at least two strokes, and was turned after each stroke, so as to make it perfectly round and press down the projecting fins which form in the first stroke at the joint between the dies. Further, by more careful individual handling of the rivets in the heating furnace, it was possible to avoid the scale which often forms on the rivets due to improper heating when handled in bulk. Whatever slight scale was left was removed by running the finished rivets through a rumbling process and, where necessary, projections were removed by grinding.



1¼″ TAPERED RIVET

FIG. 19.

Planing Ends of Chord Members.—Great care was taken to secure perfect planing of the ends of the chords. The planing was done by a Bermet Miles horizontal and vertical planer which was especially erected for this work. It is mounted on a rotary platform, and has a cutting range of 12½ ft. horizontally and 10½ ft. vertically.

The three-faced joint of the bottom chord members was obtained by facing the member first to a single plane for the full width. In this condition the chord member was assembled in the truss, and after the truss had again been taken apart the two outer thirds of the face of the chord were planed to the required bevel. About 1 in. of metal was cut away at each face, first by a roughing cut parallel to the vertical webs, and then by a horizontal finishing cut. The length

of the members had to be accurate within $\frac{1}{64}$ in. and the bevels of the faces within 30 sec. (= $\frac{1}{64}$ in. in 10 ft.).

Assembling Arch Trusses.—Complete or partial assembling of riveted trusses, or of continuous chords of pin-connected trusses, is gradually becoming the ordinary method in American practice, at least for important work. With improved facilities for assembling large trusses at the shop, the reaming and drilling of the field connections can be done more cheaply than by the formerly prevailing practice of reaming or drilling to iron templates. The assembling at the shop, moreover, insures greater accuracy and decreases the chances for errors or unforeseen difficulties, with the resulting delays and added expenses, in the field.

As the complete assembling of the arch trusses would have required a very large space, and corresponding facilities for handling the members, the Bridge Company was permitted to assemble the truss in sections of four panels, the last panel of each section being again assembled with the following three panels.

Each truss section was laid out carefully in the bridge shop yard to the correct cambered form with a transit. The members were supported by timber grillages of sufficient bearing area to prevent excessive settlements. Levels were taken each morning before any work was done to make sure that the truss section was in a perfect plane during the drilling of the holes. The members were handled with a gantry crane of 130 ft. span and 150 tons lifting capacity (Fig. 20). This crane was erected specially for this work.

As previously mentioned, all holes for the field connections were drilled to a diameter of $\frac{11}{16}$ in. from the solid, and $\frac{5}{8}$-in. bolts were used for assembling. After a truss section was assembled, the holes were drilled to full size with long high-speed drills which reached through both webs. The $\frac{5}{8}$-in. bolts were gradually replaced by larger ones as the drilling progressed, so as to keep the members and gussets always firmly tied together. Before the members were taken apart, the field connections were match-marked carefully and the marks were recorded on a chart for the use of the field force. Fig. 21 shows the special lifting device used for handling the heavy chords.

Only the best class of labor was employed on this work, and the bridge shop deserves full credit for its excellent work, which manifested itself in the accuracy and expediency with which the members

were erected in the field. The 250-ton cast-steel bearings (Fig. 22) were completely fitted together at the shop.

Angles Between Truss Members and Bevels of Joints.—In the fabrication of large riveted trusses, careful consideration must be given to the question as to whether the angles between members and the bevels of faced ends of chord members shall be made to conform to the "cambered", or to the "geometric", or some other form of the truss. The method to be used depends essentially on the desirability of reducing the secondary stresses, and therefore on the kind and system of truss.

If the angles are laid out to the geometric form of truss, that is, the form which the truss is expected to assume after completion, it is possible to reduce the secondary stresses considerably. This method, therefore, is advisable for trusses with high secondary stresses, such as cantilevers or continuous trusses.

If the angles are made to conform to the cambered form of truss, that is, the form which the truss would assume when entirely relieved of stress, the secondary stresses are, theoretically, fully developed. This method was used for the Hell Gate Bridge, as the secondary stresses are negligible, and because this method has the following important advantages:

First, the whole truss or any number of panels can be completely put together at the shop, with tight joints, and the holes for the connections can be reamed or drilled while the truss is thus assembled, with the greatest possible accuracy and least chance of errors.

Second, when the truss members are erected in the field, the holes of the connections should match perfectly, and the joints should be tight, without initial bending of the members. The riveting of the connections can start at once, if desired, or the holes can be filled temporarily with tight-fitting drift-pins and bolts to prevent motion of the ends of the member during the deformation of the truss. This secures the greatest safety and expediency in erection.

11.—Erection of Hell Gate Bridge.

Method of Erection.—As is necessary in the case of a large bridge which has few or no precedents, the question of erection was given thorough consideration by the Consulting Engineer, who prepared general erection plans and stress sheets before the design was finally

FIG. 20.—ASSEMBLING ARCH TRUSSES AT SHOP, FOR HELL GATE BRIDGE.

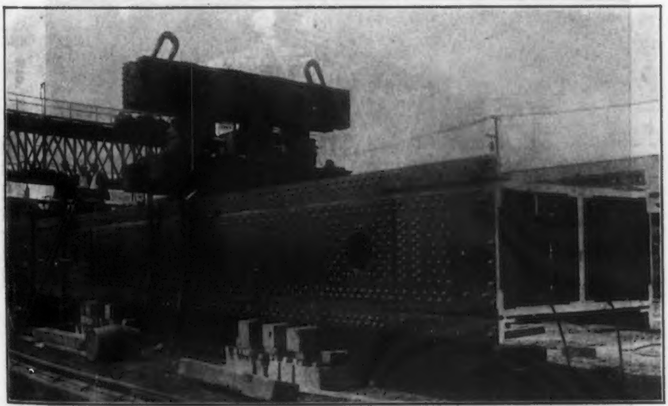


FIG. 21.—BOTTOM CHORD SECTION WITH LIFTING DEVICE, HELL GATE BRIDGE.

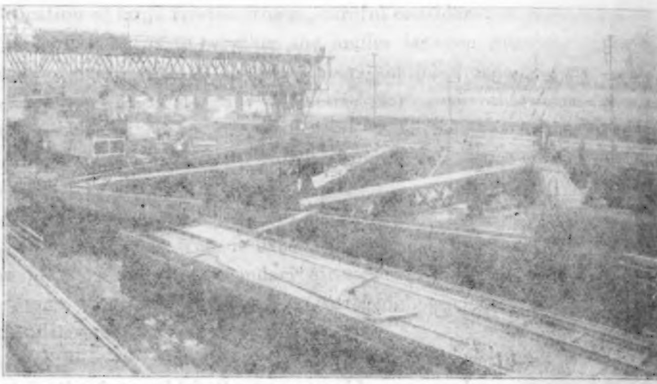


FIG. 20.—ASSEMBLING ARCH TRUSSES AT SHOP, FOR HELL GATE BRIDGE.

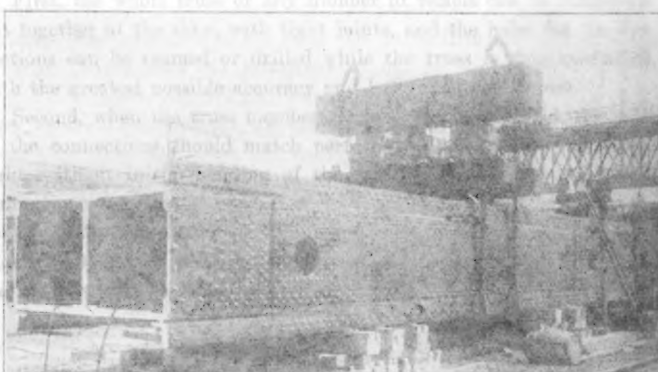


FIG. 21.—BOTTOM CHORD SECTION WITH LIFTING DEVICE, HELL GATE BRIDGE.

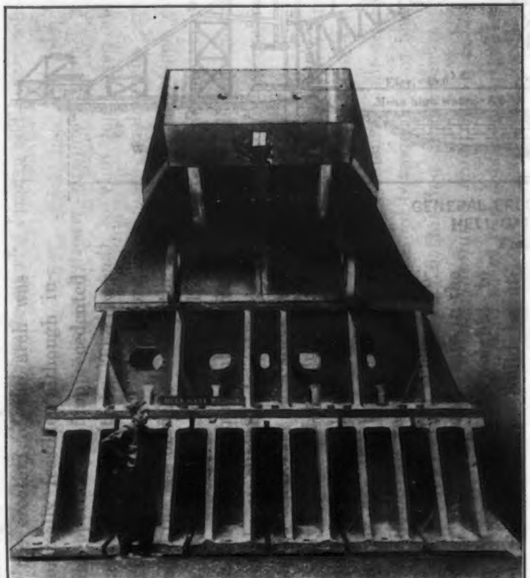


FIG. 22.—ASSEMBLING OF 250-TON ARCH BEARING, HELL GATE BRIDGE.

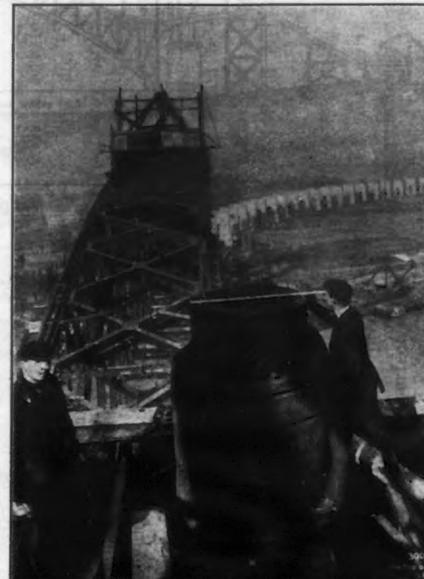


FIG. 23.—3 000-TON HYDRAULIC JACK, HELL GATE BRIDGE.

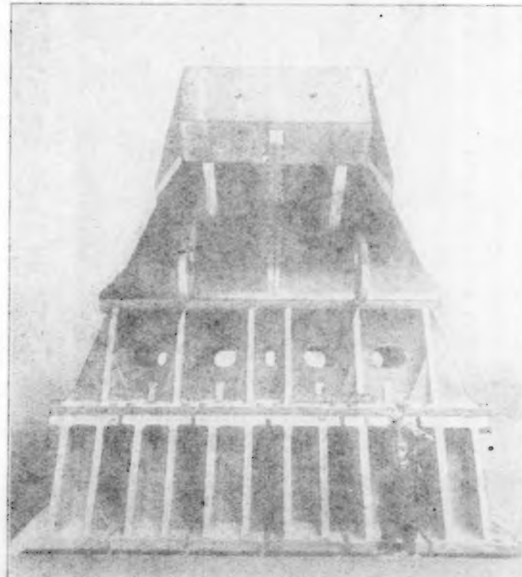
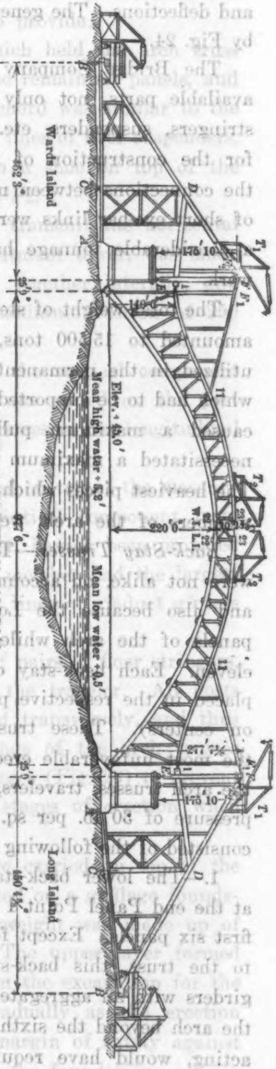


Fig. 22.—Assembling of 200-Ton Arch Bearing, Hell Gate Bridge.

Fig. 23.—2 000-Ton Hydraulic Jack, Hell Gate Bridge.

decided upon. These plans indicated that the erection of the arch was economically feasible, although involving operations of unprecedented character and magnitude. The river conditions, as outlined heretofore, excluded the use of falsework in the river, except for a very short distance from each abutment. Erection on the cantilever principle, with the use of temporary back-stays, was the only practicable alternative. However, as natural anchorages in solid rock for the back-stays were not available at reasonable depth below the surface, the scheme provided for artificial anchorages in the form of huge counterweights to which the back-stays could be attached. To transmit the horizontal pull of the back-stays to the abutment towers, where it could find resistance, struts had to be provided along the ground surface between the counterweights and the towers. To render this method economical, it provided for the temporary use of parts of the permanent bridge in the back-stays.

The general features of this scheme were adopted by the erectors, the American Bridge Company, with whom the responsibility for the erection properly rested. The erectors' plans, which were prepared with a view to secure perfect safety during erection, were approved by the Consulting Engineer after thor-



GENERAL ERECTION DIAGRAM
HELL GATE BRIDGE
Fig. 24

ough examination and independent calculation of all erection stresses and deflections. The general scheme of erection, as used, is illustrated by Fig. 24.

The Bridge Company resorted to a very skillful utilization of available parts, not only of the arch bridge proper, such as floor stringers, suspenders, etc., but also of the plate-girder approaches, for the construction of the back-stays and counterweights. Only the connections between members, some light bracing, and a number of short eye-bar links were made of extra material, and even of this a considerable tonnage had been used previously on other erection work.

The total weight of steel in the back-stays, for both sides together, amounted to 15 500 tons, of which only about 2 300 tons are not utilized in the permanent structure. The total weight of the arch which had to be supported by the back-stays amounted to 14 500 tons, caused a maximum pull of 6 500 tons in each back-stay, and necessitated a maximum counterweight of 5 300 tons on each side. The heaviest pieces which had to be lifted as units, the bottom chord members of the arch, weighed 180 tons.

Back-Stay Trusses.—The back-stays on the two sides of the river were not alike, on account of different configuration of the surface, and also because the Long Island back-stay had to carry twelve panels of the arch while the Wards Island back-stay carried only eleven. Each back-stay consisted of two separate back-stay trusses placed in the respective planes of the two arch trusses (60 ft. apart on centers). These trusses were designed to resist safely, under the most unfavorable erection conditions, their own weight, that of the arch trusses, travelers, and other erection equipment, and a wind pressure of 30 lb. per sq. ft. of exposed area. Each back-stay truss consisted of the following essential parts (Fig. 24):

1.—The lower back-stay chord, *BD1*, which held the arch truss at the end Panel Point 1 of the top chord during the erection of the first six panels. Except for the short eye-bar links at the connection to the truss, this back-stay chord consisted of two lines of plate girders with an aggregate net section of 316 sq. in. The erection of the arch beyond the sixth panel, with only the lower back-stay chord acting, would have required an excessively large section for this

back-stay and considerable extra material in the top chord of the arch trusses. It was necessary, therefore, to provide:

2.—An upper back-stay chord, *DF*11, which held the arch truss at Panel Point 11 during the erection of the remaining panels, and the closure of the arch. Part, *DF*, of this chord was similar to the lower chord, and part, *F*11, consisted of three lines of floor suspenders. Both parts of this chord were connected to a shoe on top of the column, *EF*, by short eye-bar links, *FF*1 and *FF*2.

3.—The bottom strut, *AB*, which had to transmit the horizontal component of the back-stay chord to the abutment tower, was made up of four lines of floor stringers, braced together laterally, and supported by timber grillages. It had a maximum section of 290 sq. in.

4.—The vertical post, *CD*, had to carry the vertical component from the lower tension chord. The chord was seated on the post by a roller nest in order to prevent serious stresses in the longitudinal bracing between posts.

5.—The vertical post, *EF*, which was seated on top of the masonry tower at track level had to transmit the vertical component from the upper-tension chord. It acted as a rocker, having been provided with pin bearings at top and bottom. This member had the largest section, 348 sq. in., and was made up of four lines of viaduct girders with their bracing.

All other vertical posts, made up mostly of pairs of floor stringers, carried only the weight of the chords and the traveler. All posts were well braced in pairs longitudinally and transversely, and thus formed rigid towers and bents. The bearings of the arch trusses were provided with temporary eye-bar anchorages (Fig. 26) to prevent sliding on the skewbacks during the early stages of erection when the reaction had the steepest inclination.

Counterweights.—The counterweights were carried on top of the rear ends of the back-stay trusses, which rested on a grillage foundation. On the Long Island side the counterweight was made up of three layers of viaduct girders (Fig. 25). The upper layer formed a box which was filled with earth taken from the excavation for the bottom strut. The earth was filled in gradually as the erection proceeded so that there was always ample margin of safety against uplift and at the same time safe pressure on the foundation.

The counterweight on the Wards Island side was similar, except that, on account of lack of sufficient earth excavation, the necessary weight was made up by additional viaduct girders and flooring I-beams.

To allow free expansion and contraction of the back-stay trusses, and thus prevent large temperature stresses, the ends of the back-stay trusses under the counterweights rested on roller bearings. Provision was also made to raise the counterweights to proper height by eight 500-ton hydraulic jacks (four under each truss) so as to offset settlements of the foundations. These jacks, having been gauged for pressure, served at the same time as a check for the weight of the earth fill.

3 000-Ton Hydraulic Jacks.—Adjustment of the arch trusses in height was required at various erection stages. For this purpose a powerful hydraulic jack (Fig. 23) was placed at the top of each of the four erection posts, *EF* (Fig. 24). Each jack had a lifting capacity of 3 000 tons under a water pressure of about 5 000 lb. per sq. in. The cast-iron jack plungers had a diameter of 39 in. and a maximum stroke of 26 in. The cylinders were of cast steel, and each one was tested at the United States Government testing plant to its full capacity. When operated, the plunger acted against the cast-steel shoe on top of the post, raising or lowering it as desired, and thereby raising or lowering the arch trusses or rather revolving them around their bearings. As the shoe was raised or lowered, shim plates were inserted or removed from between the shoes and their original bearings on the post, so that the jacks could be relieved after the jacking operation was completed.

General Erection Procedure.—The diagrams on Plate XXXIII illustrate the general procedure and the progress of erection. The erection was to be carried out simultaneously from both sides of the river, but, on account of delays in the construction of the deep foundation of the Wards Island tower, erection on that side was started 4½ months later than on the Long Island side. It was timed, however, so that the two halves of the arch were completed simultaneously. The erection of each half span proceeded in the following order (Fig. 24):

1.—Placing of compression chords of back-stay, on the previously graded surface, with a 60-ton locomotive crane.

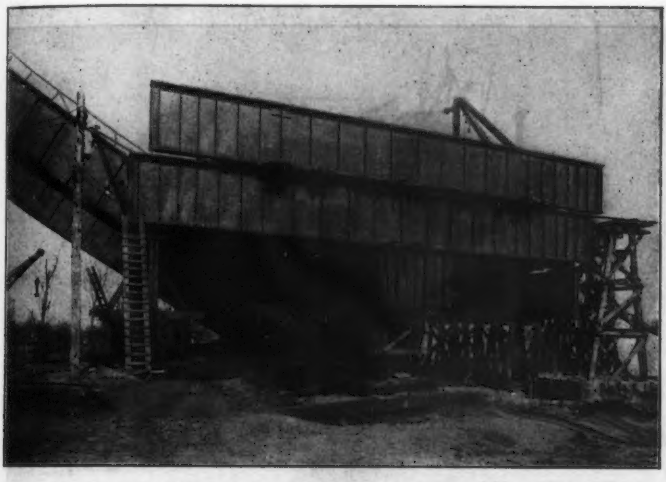


Fig. 25.—Long Island Back-Stay Counterweight, for Hell Gate Bridge.

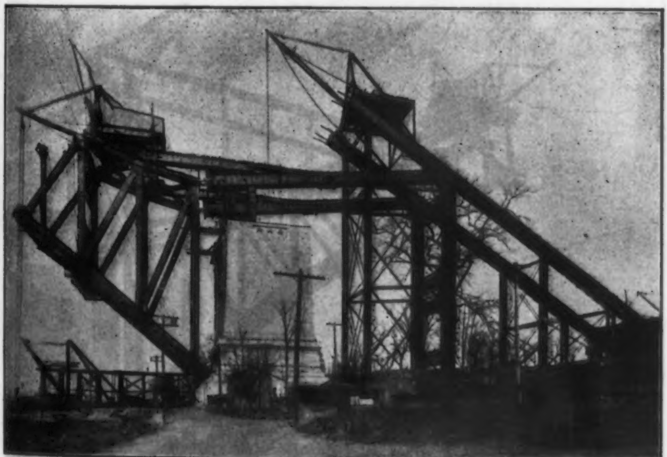


Fig. 26.—Hell Gate Bridge: Erection Stage, April 1st, 1915, Long Island Side.

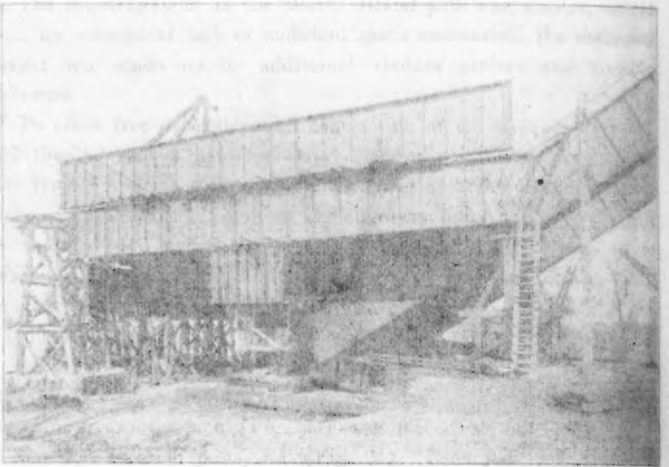


FIG. 25.—LONG ISLAND BACK-STAY COUNTERWEIGHT, FOR HELL GATE BRIDGE.

FIG. 26.—HELL GATE BRIDGE: ERECTION STAGE, APRIL 1ST, 1915, LONG ISLAND SIDE.

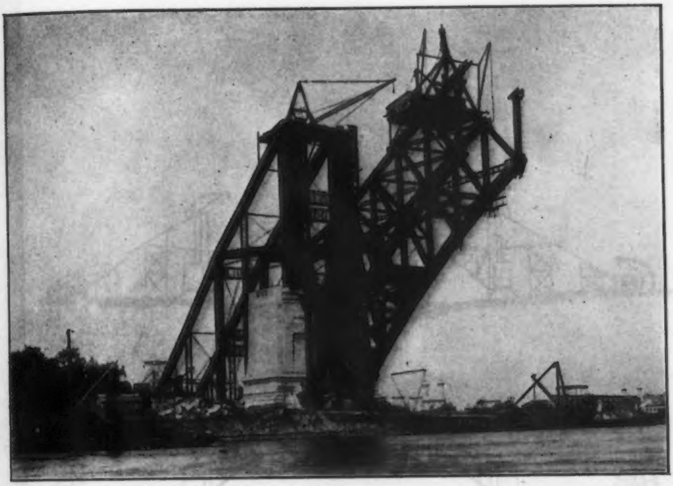


FIG. 27.—HELL GATE BRIDGE: ERECTION STAGE, AUGUST 18TH, 1915, WARDS ISLAND SIDE.

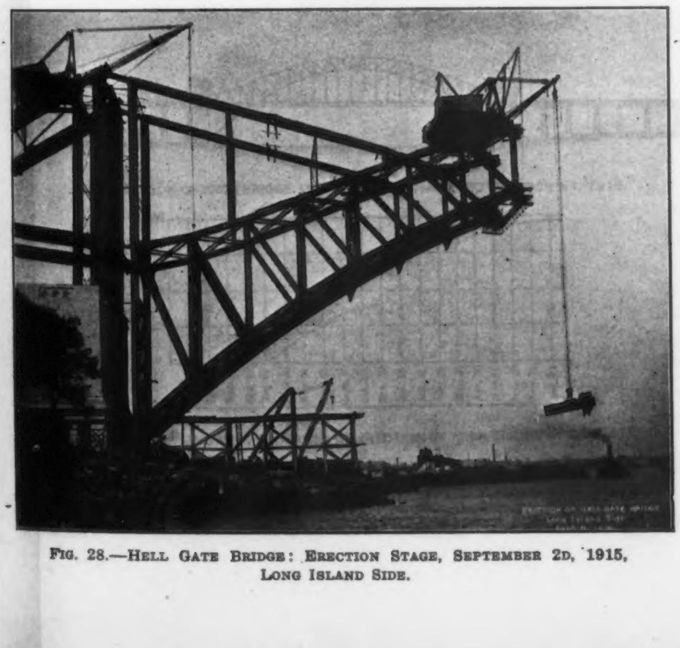


FIG. 28.—HELL GATE BRIDGE: ERECTION STAGE, SEPTEMBER 2D, 1915, LONG ISLAND SIDE.

Fig. 27.—Hell Gate Bridge: Erection Stage, August 18th, 1915, Wards Island Side.

Fig. 28.—Hell Gate Bridge: Erection Stage, September [illegible], 1915, Long Island Side.

PLATE XXXIII.
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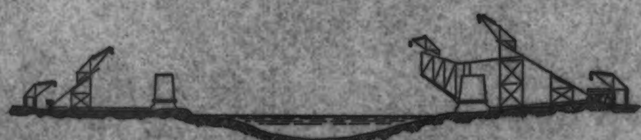
PROGRESS DIAGRAM OF ERECTION OF HELL GATE BRIDGE.

DIAGRAM SHOWING GENERAL DIMENSIONS OF ARCH BRIDGE AND BACK-STAYS

November 30, 1914.

January 31, 1915.



March 31, 1915.

May 31, 1915.

July 31, 1915.

September 30, 1915.

October 15, 1915.

October 21, 1915.

December 31, 1915.

ERECTION OF ARCH BRIDGE AND VIADUCTS COMPLETED OCTOBER 31, 1916.

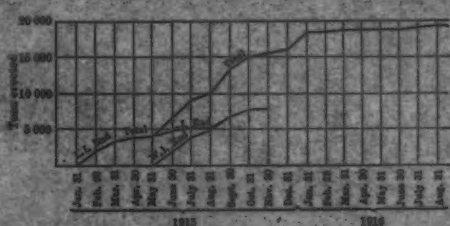


DIAGRAM SHOWING PROGRESS OF ERECTION OF ARCH BRIDGE BY TONNAGE

2.—Erecting bottom layer of counterweight girders and on top of these the back-stay traveler, T_1, with a 30-ton stiff-leg derrick placed back of the counterweight.

3.—Erecting lower part, *BD*1, of back-stay with the back-stay traveler, T_1, which moved along the top of the back-stay.

4.—Erecting arch traveler, T_2, on top of lower tension chord of back-stay by the traveler, T_1.

5.—Setting of bearings for arch bridge, and erecting first six panels of arch trusses, bracing, and floor-beams, by the traveler, T_2, which moved along the top chord of the arch. At the same time the traveler, T_1, completed the erection of the upper part, *DF*, of the back-stay, and portion of the tie, *F*-11, by advancing on top of the back-stay as far as the point, *F*. The front portion of the tie, *F*-11, was erected by a derrick mounted on the rear end of the traveler, T_2.

6.—Connecting the upper chord, *F*-11, to Panel Point 11 of the arch, and raising the point, *F*, with the hydraulic jacks until the lower chord, *D*-1, was relieved of its stress and disconnected.

7.—Continuing erection of arch trusses and bracing to the center (eleven panels on Wards Island side and twelve panels on Long Island side) leaving a small gap between the two ends (at bottom chord point 22 *WI*).

8.—Lowering Points *F* with the hydraulic jacks until the trusses became self-supporting three-hinged arches.

9.—Moving the travelers, T_2, back to Panel Point 11, where they started the removal of the forward stay and the erection of the floor suspenders and floor-beams, proceeding again toward the center. (The stringers could not be erected in this operation because they formed the compression chords of the back-stays and were at that time not yet dismantled.)

10.—Dismantling of back-stays and back-stay travelers in the reverse order in which they had been erected.

11.—Erecting stringers, railings, and floor bracing of arch by the traveler, T_2, from the center toward the ends. The end panels of the floor were erected, and the travelers, T_2, were dismantled later by a derrick set up at the end of the top chord.

12.—Connecting up of top chord and diagonals in the center panel so as to transform the trusses into two-hinged arches.

13.—Placing of concrete flooring, ballast, and tracks, after the riveting of the arch trusses had been completed.

Erection of Truss Members.—The simplicity and uniformity in the design of the various truss panels greatly facilitated and expedited the erection. The typical procedure in erecting a panel of the arch was as follows: The two bottom chords were raised into position, one at a time, the arch traveler standing with its front truck at the end of the previously completed panel. As soon as a chord was in place, the connection at the rear end was made with bolts and drift-pins, and then the falls were released and the chord was allowed to cantilever out. The gusset and splice-plates at the front end of the chord had previously been bolted to the chord on the ground.

Next, the diagonals were raised and connected at both ends, then followed the laterals between bottom chords, the vertical posts, top chord members, and last the sway-bracing between the posts and the top laterals, only one member being raised at a time. On account of the great weight of the chord members it was considered that better progress could be made by raising them individually, instead of in pairs, and therefore the traveler had been designed with a single central boom.

After the completion of a panel, the traveler was moved forward to the next panel point and blocked. The time consumed for the erection of a panel decreased as the erection advanced toward the center, as the erection gangs became more experienced and the members decreased in weight. It took about 3 weeks to erect the end panel on the Long Island side, and the tenth panel on the Wards Island side was completely put up in 7½ hours. The members of the center panel were raised jointly by both arch travelers, the latter standing with their front trucks at Points 21 (Fig. 29). Owing to the accuracy of the shop work, and the careful assembling of the trusses at the shop, no serious difficulties or delays were experienced in making the connections in the field, notwithstanding the unusual number and dimensions of gusset and splice-plates and the fact that no clearance had been allowed for entering parts.

Deflections and Adjustments of Arch Trusses During Erection.—During their erection, the arch trusses passed through the following four principal and distinct static conditions:

1.—Cantilever condition. During erection of first six panels, truss held at end of top chord by lower back-stay (Figs. 26 and 27).

2.—Cantilever condition. During erection of remaining panels, truss held at top chord Point 11 by upper back-stay (Figs. 28 and 29).

3.—Three-hinged arch condition. Back-stays released and trusses connected at bottom chord Point 22 Wards Island side, which acted as hinge, top chord 23-23, and diagonal 23 Wards Island side—22 Long Island side not connected at 23 Wards Island side (Fig. 30). Arch left in this condition until all steelwork had been erected.

4.—Final or two-hinged arch condition. All steelwork erected and all members of the center panel fully connected.

Each transformation from one to the next of these principal static conditions marked a critical erection operation. The first two operations required adjustments of the arch trusses, and to determine the amounts of these adjustments it was necessary to calculate the deflections of certain points of the trusses during erection.

To obtain a clear illustration of the elastic deformation of the trusses during erection, and for the purpose of comparison with observations in the field, complete deflection diagrams for the various erection stages were prepared. The diagrams for the principal stages are shown on Plate XXXIV. The deflections of the arch trusses during the cantilever conditions were due, first, to the elastic deformation of the trusses themselves from dead load and weight of traveler, etc.; second, to the elastic deformation of the back-stays from the same loads; and, third, to the change in length of the members from temperature changes. These deflections were very considerable. The total deflection of the bottom chord Point 22, Wards Island side, for instance, under the extreme cantilever condition and at normal temperature (60° Fahr.), was theoretically as follows:

	Wards Island side.	Long Island side.
Vertical downward deflection....	20.6 in.	26.6 in.
Horizontal outward deflection....	7.6 in.	8.1 in.

At a temperature of 30° above normal, these points would have moved 1.6 in. farther out, which would have resulted in an overlap of the two ends of about 19 in. In order that the ends of the two arms should clear each other and that connection at the center could be made by lowering the trusses, it was necessary, therefore, to erect each half in a raised position (revolved around its end bearing).

This position was determined so that, at a temperature of 30° Fahr. above normal, which was considered the extreme high temperature at which the connection might have to be made, there would still be an opening between the ends of the two arms of about $1\frac{1}{2}$ in., to allow for discrepancy between theoretical and actual deflections. That corresponded to an opening of 4.8 in. at normal temperature and 8.1 in. at 30° below normal. The raised position of the trusses was established by making the eye-bar links of the back-stays at Points *F* shorter than their geometric lengths. Link *FF*1 was shortened by $12\frac{9}{16}$ in. on the Long Island side and by $11\frac{1}{16}$ in. on the Wards Island side, and *FF*2 by $1\frac{3}{8}$ in. on both sides. All other members of the back-stays were made to their true geometric length, and the pin at Point *F* was assumed to be raised to its normal or geometric elevation. The position for that extreme cantilever condition (Stages 12 and 12*a*) and for normal temperature is shown (by full lines) in the third diagram on Plate XXXIV. The positions of the various panel points are given with reference to the geometric position of the arch trusses (shown by dotted lines).

To determine the required range of adjustment by the hydraulic jacks at Points *F*, that is, the amount by which Points *F* had to be lowered by the jacks to close the opening at the center and transform the trusses into a self-supporting three-hinged arch, it was necessary to calculate the position of the trusses for the latter condition. That position is shown in the fourth diagram (Stage 13) on Plate XXXIV. This diagram shows also the intermediate position (dash and dotted line) which the trusses assumed when the ends of the two arms just came into contact, without transmitting any stress. From the commencement of lowering to this intermediate stage the movement of the trusses was only a downward rotation around their bearings; during the remainder of the movement the trusses changed their elastic form, rising in the center about $1\frac{3}{4}$ in. and sagging at the quarter points about 3 in.

To determine the movements taking place during the operation of releasing the lower back-stay, it was necessary to calculate the positions of the trusses immediately before and after this operation. These positions are shown in the first and second diagrams on Plate XXXIV (Stages 6 and 7). The position at Stage 7 is obtained from the position at Stage 6 by raising Point *F* from an initial low position

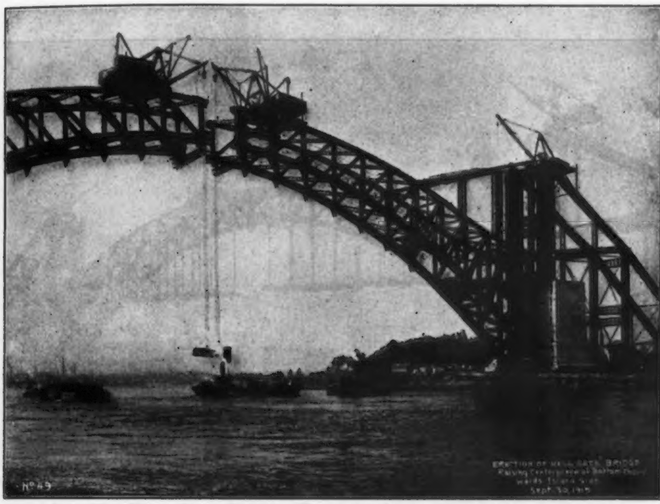


Fig. 29.—Erection Stage, September 30th, 1915. Raising Center Piece of Bottom Chord, Hell Gate Bridge.

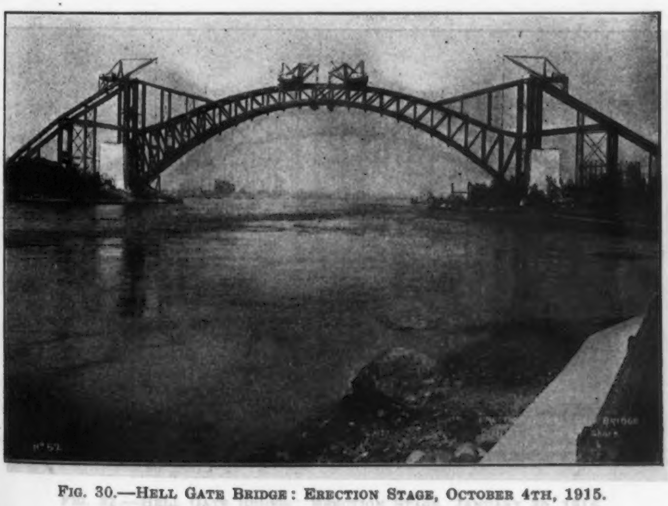


Fig. 30.—Hell Gate Bridge: Erection Stage, October 4th, 1915.

FIG. 29.—ERECTION STAGE, SEPTEMBER 30TH, 1915. RAISING CENTER PIECE OF BOTTOM CHORD, HELL GATE BRIDGE.

FIG. 30.—HELL GATE BRIDGE: ERECTION STAGE, OCTOBER 4TH, 1915.

FIG. 31.—HELL GATE BRIDGE: ERECTION STAGE, NOVEMBER 1ST, 1915.

FIG. 32.—HELL GATE BRIDGE: ERECTION STAGE, JANUARY 3D, 1916.

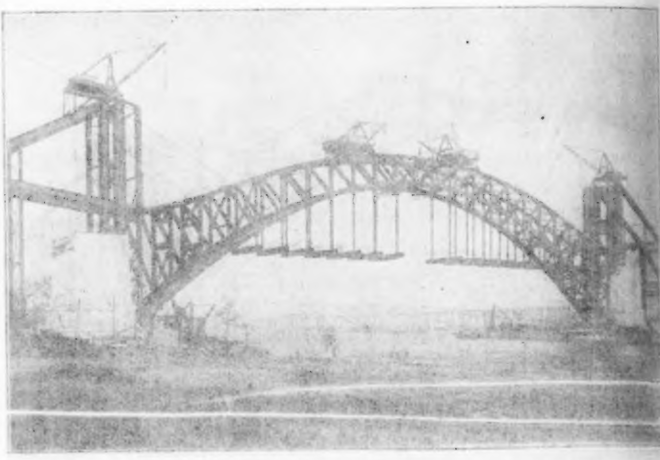


Fig. 31.—Hell Gate Bridge: Erection Stage, November 1st, 1915.

Fig. 32.—Hell Gate Bridge: Erection Stage, January 3d, 1916.

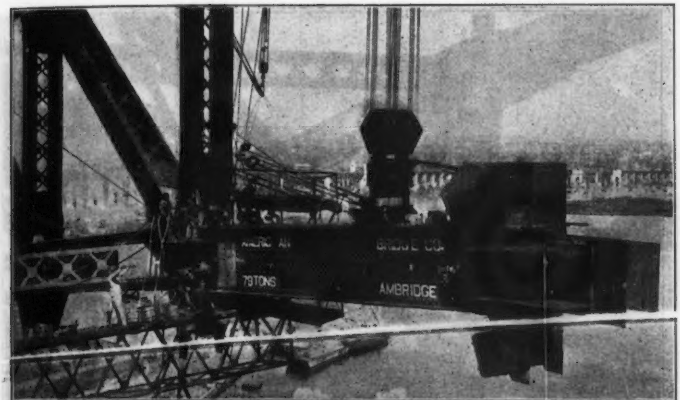


FIG. 33.—CONNECTING BOTTOM CHORD MEMBER, HELL GATE BRIDGE.

FIG. 34.—CONNECTING CENTER PANEL OF BOTTOM CHORD, SEPTEMBER 30TH, 1915, HELL GATE BRIDGE.

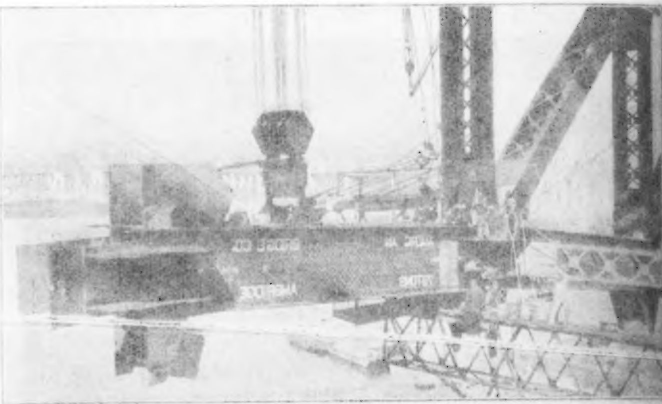


Fig. 33.—Connecting Bottom Chord Member, Hell Gate Bridge.

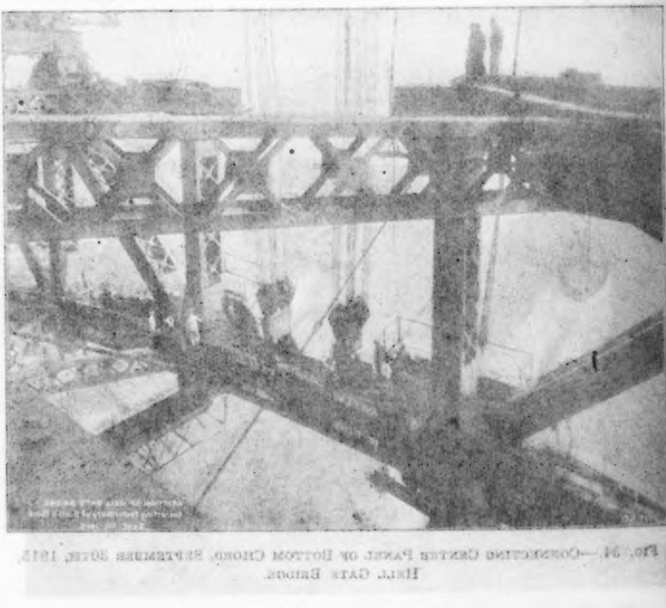


Fig. 34.—Connecting Center Panel of Bottom Chord, September 30th, 1915, Hell Gate Bridge.

FIG. 35.—HELL GATE BRIDGE: CENTER PANEL COMPLETELY ERECTED, OCTOBER 4TH, 1915.

FIG. 36.—HELL GATE BRIDGE: TRANSPORTATION OF BOTTOM CHORD MEMBERS.

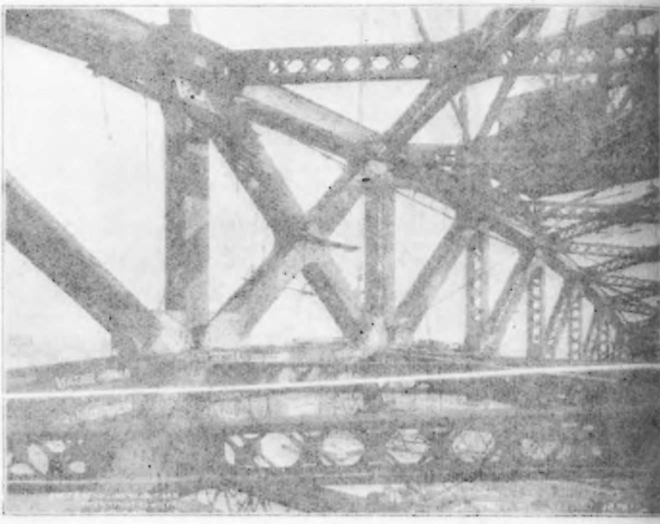


Fig. 25.—Hell Gate Bridge: Center Panel Completely Erected, October 4th, 1915.

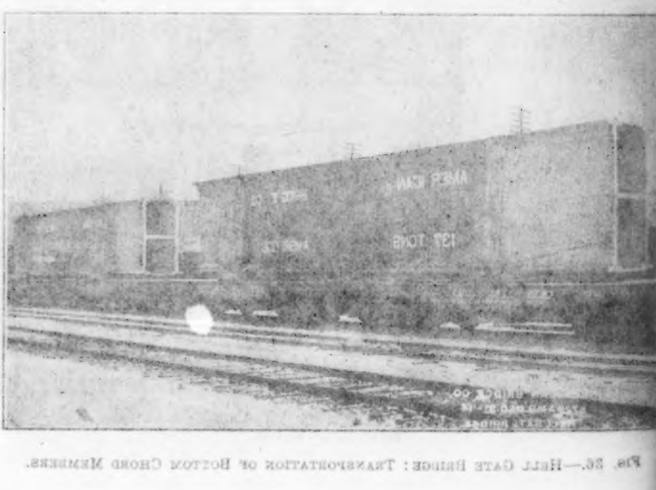


Fig. 26.—Hell Gate Bridge: Transportation of Bottom Chord Members.

PLATE XXXIV.
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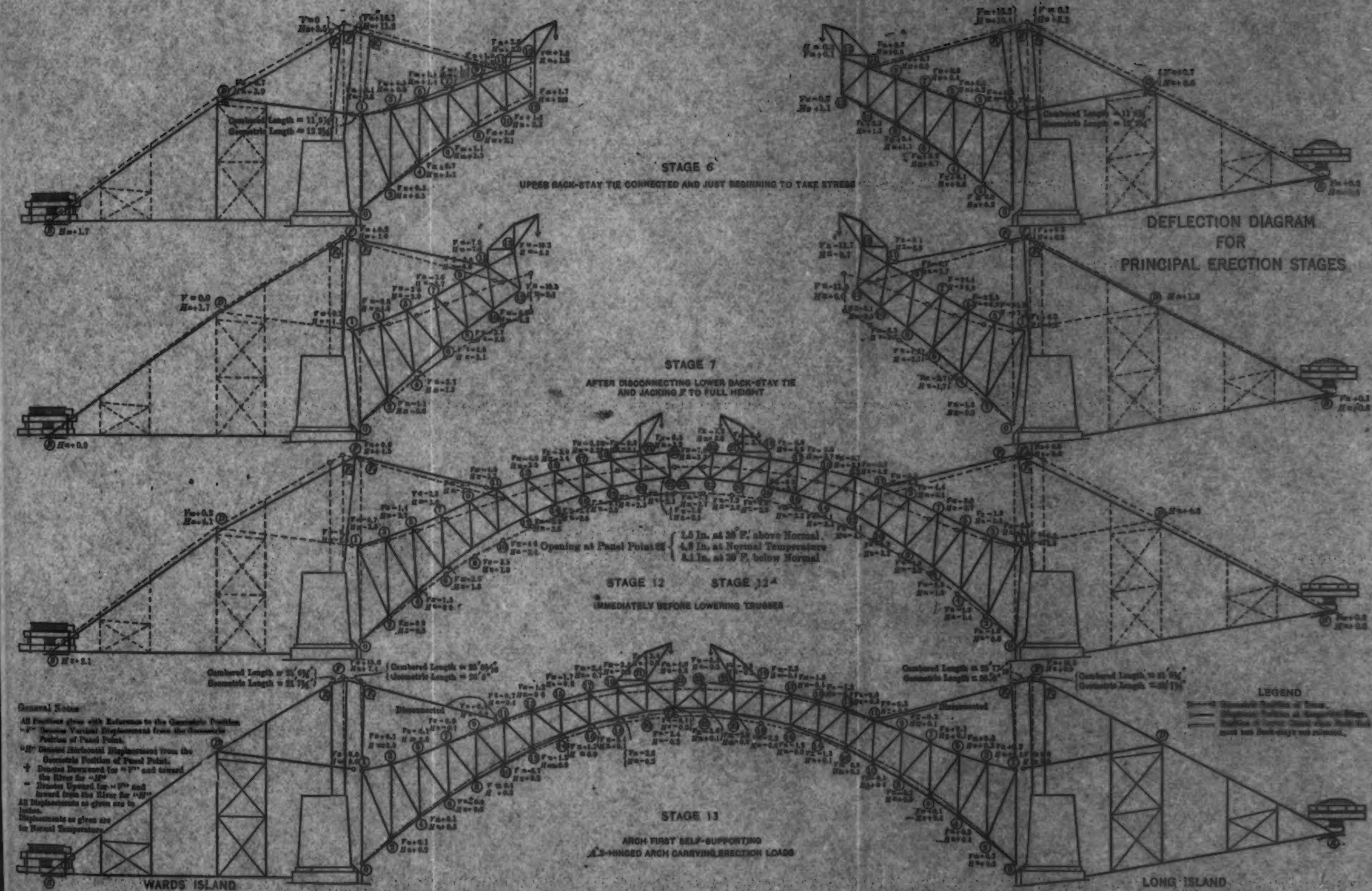


to the normal elevation required for Stage 12. To keep the amount of jacking within the range required for the closing operation, it was found necessary to make the eye-bar link, *l-g*, of the lower back-stay shorter than its geometric length by 5½ in. on the Long Island side and by 4½ in. on the Wards Island side.

During the first part of the movement from Stage 6 to Stage 7, during which the upper back-stay gradually took its full stress, the trusses changed their elastic form; during the second part, after the lower back-stays had been disconnected, the trusses merely revolved around their bearings. The intermediate stage is not shown in the deflection diagrams. The total vertical movement of Panel Point 11 during this operation was 9.6 in. on the Long Island side and 9.9 in. on the Wards Island side. It is interesting to note, also, the very considerable horizontal deflection of Points *F* at the top of the back-stay column during this operation, amounting to 10.4 in. on the Long Island side and 11.2 in. on the Wards Island side. Fig. 37 shows the theoretical movements of the hydraulic jacks and the positions of the pins, *F*, at the various critical stages.

Jacking Operation for Change from Lower to Upper Back-Stay.— When Erection Stage 6 (Plate XXXIV) had been reached, the upper back-stay was connected to the truss at Panel Point 11, with a slight play between the connecting pin and its bearing on the gusset-plates of the truss. The shoes at *F* were then jacked up until the pin at 11 got a firm bearing and the upper back-stay began to take stress (Fig. 37*a*). Jacking was then continued until the upper back-stay had taken full stress and the lower back-stay could be disconnected (Fig. 37*b*). After this the shoes were further raised to their normal elevation (Fig. 37*c*), corresponding to the required position of the trusses for Stage 7. The calculated total raising of the shoes was 22½ in. The shims between the shoe and their original bearings on the post were inserted as fast as the jacking proceeded, and, after completion of the jacking, the jacks were released.

Actually, the procedure on the Long Island side differed somewhat in that the jacking operation was discontinued after the upper tie had taken about 75% of its stress. Erection was then continued until the seventh panel had been placed, after which the jacking operation was completed. The reason for this procedure was that, in changing from Stage 6 to Stage 7, the foundation pressure under the counter-

weight would have increased from 2.0 to 3.9 tons per sq. ft. This sudden increase might have caused excessive settlement, as the ground was comparatively soft. By adding the seventh panel and moving the traveler to Panel Point 15 before completing the jacking, it was possible to keep the foundation pressure within 3 tons.

A complete record of this jacking operation for both sides is given in Tables 4 and 5. It shows a remarkable coincidence between the jacking heights as calculated and as observed. The "effective" jacking height, that is, the amount of jacking from the moment the upper back-stay started to take stress until the stress in the lower back-stay became zero, was actually 11 in. (north truss) and 11⅜ in. (south truss), on the Long Island side, and 13 in. (north truss) and 13 in. (south truss), on the Wards Island side, as compared with 10⅞ in.

TABLE 4.—RECORD OF JACKING FOR CHANGING

Date.	Time.	Remarks.	Height of Shims. Observed: N. Truss. Inches.	Observed: S. Truss. Inches.	Calc. Inches.	Percentage of effective jacking completed.
Aug. 28.	....	STAGE 6.—Lower stay fully stressed..........	3⅝	3⅝	....	....
Aug. 31.	....	Slight jacking to take out slack in upper stay.	4¼†	3⅞†	6⅜	0
		Continued jacking and stopped at............	6	5⅞	....	14
Sept. 2.	8.45	Started jacking at..............................	5⅞	5⁹⁄₁₆	....	13
	9.20	Continued jacking..............................	14½	14⅛	....	77
	9.33	Continued jacking..............................	16⅜	15¾	....	90
	9.38	Continued jacking..............................	17⅛	16⅝	....	97
	9.41	Continued jacking and stopped at...........	17⅝	17⅛	19½	100
	9.46	Jacks lowered to bearing on shims...........	16⅜	16¼	....	92
Sept. 8.	....	Raised jacks—7 panels erected	17	16⅛	....	97
		Continued jacking to remove pins in lower stay	17¼	17⅛	19½	100
		Total effective jacking from 0 to full stress in upper stay..................................	13	13¼	13⅛	....
		Continued jacking after removing pins in lower stay...............................	22⁷⁄₁₆	22⅜	22¼	....
Sept. 9.	....	Raised jack to level up pins over jacks and at Panel Point No. 11.........................	22⅝	21⅛	22½	....

No allowance made for friction. † Interpolated, not observed. ‡ Stress falling rapidly.

(Long Island side) and $13\frac{1}{8}$ in. (Wards Island side), respectively, as calculated. The tables also show a close agreement between the tension stresses in the eye-bar links of the upper and lower back-stays as calculated and as actually observed in the field by extensometer measurements. This proves the practicability of such measurements and their value as a means of checking stresses for similar operations. The discrepancy between the calculated jack pressures and the values observed on the pressure gauge is due to the friction between the plunger and the cylinder.

Jacking Operation for Closing Arch.—The closing of the trusses at the center and their transformation from cantilevers into three-hinged arches proceeded as follows: As the Erection Stages 12 (Wards Island side) and 12*A* (Long Island side) (Plate XXXIV) had been

BACK-STAYS—HELL GATE ARCH—WARDS ISLAND END.

JACK PRESSURE.		TENSION IN LOWER STAY (EYE-BAR LINKS).			TENSION IN UPPER STAY (EYE-BAR LINKS).			Remarks.
Meas.	Calc.*	Observed: N. Truss.	Observed: S. Truss.	Calc.	Observed: N. Truss.	Observed: S. Truss.	Calc.	
Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	
0	0	19 500	19 950	18 160	0	0	0	Before any jacking, 6 panels erected.
....	....	19 500	19 950	18 160	0	0	0	Upper stay begins to act.
500	220	17 100	17 000	16 200	1 050	1 950	1 050	In this condition until Sept. 2d, 1915.
500	200	....	....	....	....	....	....	Traveler moved off jacks before starting.
1 300	1 200	11 400‡	....	4 300	....	....	....	Extensometer readings in lower stay.
1 500	1 400	....	2 700	1 900	....	....	....	Extensometer readings in lower stay.
1 700	1 500	900	....	600	....	....	....	Extensometer readings in lower stay.
1 725	1 550	....	150	0	....	....	7 500	Lower stay eye-bars tested and found slack—STAGE 7.
1 600	1 430	2 400	2 400	1 500	7 650	8 060	8 500	In this condition till Sept. 8th.
1 900	1 660	....	....	600	....	....	8 960	
2 000	1 710	150	....	0	10 200	10 670	9 250	Pins removed to disconnect lower stay.
....	....	....	....	....	....	....	....	
2 050	1 710	....	....	....	....	....	....	
....	....	....	....	....	....	....	....	STAGE 8.—Jacking completed.

reached, the jack plungers were brought to bear against the shoes at Points *F*, which were in the position shown in Fig. 37*c*, and lifted them slightly so that the removal of the shim plates from under the shoes could be started. The jacks were then lowered until the bottom chords came into contact at Point 22, Wards Island side. After lowering a little farther, so as to produce a slight initial stress in the center chord, jacking was stopped. The connection was then made at 22, Wards Island side, with bolts and drift-pins. The jacks were then lowered farther, until the upper back-stay tie was relieved of stress (Fig. 37*d*).

Table 5 gives the record of this operation, and shows, again, a close agreement between the calculated and actual "effective" jacking heights, that is, the amount of jacking from the moment the ends of the two arms came to bear to the moment when the stress in the back-stays became zero. This height was actually 15 in. at both trusses on the Long Island side and $12\frac{1}{8}$ in. (north truss) and $12\frac{5}{16}$ in. (south truss) on the Wards Island side, as compared with $14\frac{3}{4}$ in. (Long Island side) and $12\frac{3}{16}$ in. (Wards Island side), respectively, as calculated. The opening at the center, before lowering started, was actually $4\frac{9}{16}$ in. at a temperature of 56° Fahr., as compared with $4\frac{3}{4}$ in. as calculated for the normal temperature of 60° Fahr. Panel Points 22 were found to be on an average 1 in. lower than the theoretical position, which is a very small deviation for such a great span. A difference in temperature of 10° would have been sufficient to produce that difference in elevation.

Weather conditions were very favorable for the closing operation, the sky having been covered and the temperature nearly constant at 56° Fahr. Great care had to be exercised to bring the ends of the two arms into perfect contact, and, after that, it was important to insure the simultaneous, proportionate lowering of all four jacks, as otherwise serious shearing stresses might have been caused in the center connection. This was secured by a telephone system through which constant communication was kept between the men in charge at the tops of the erection towers, where the jacks were operated, at the center of the arch, and at the main field office.

During the release of the back-stays the foundation pressure under the counterweights would, without reduction of the latter, have been gradually increased from 2.0 to 7.8 tons per sq. ft. This might have

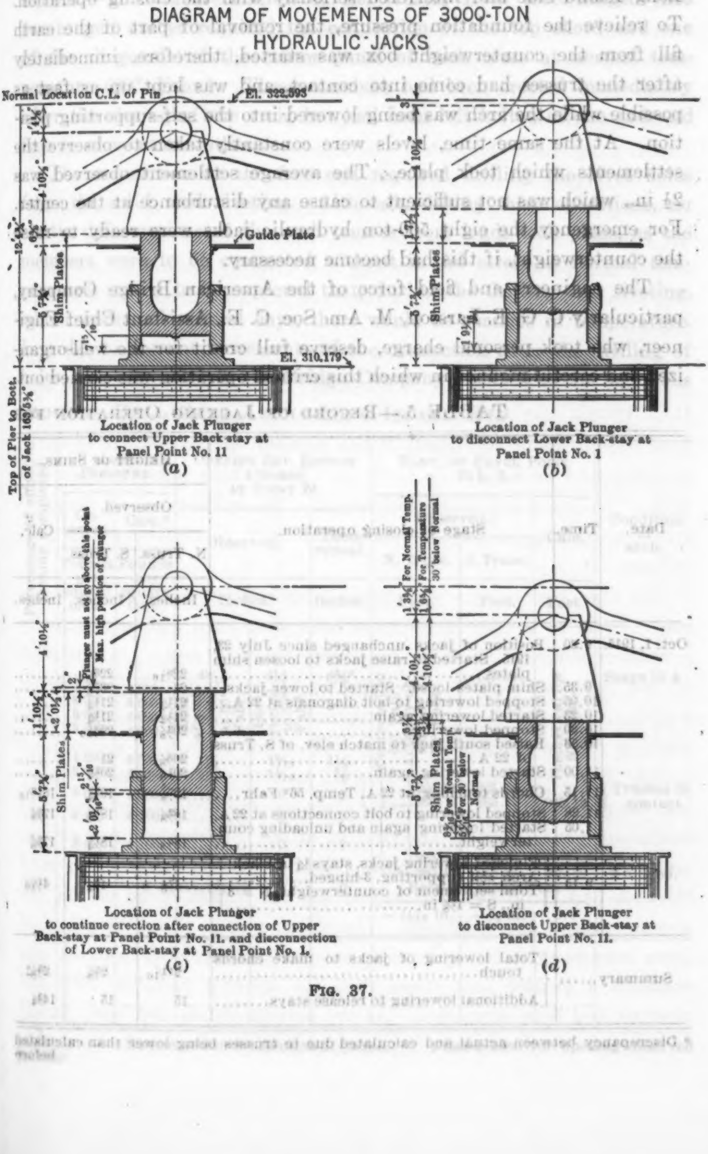


DIAGRAM OF MOVEMENTS OF 3000-TON
HYDRAULIC JACKS
Normal Location C.L. of Pin
El. 322.593'
Guide Plate
Shim Plates
El. 310.179'
Top of Pier to Bott. of Jack 16'5⅝"
Location of Jack Plunger
to connect Upper Back-stay at
Panel Point No. 11
(a)
Location of Jack Plunger
to disconnect Lower Back-stay at
Panel Point No. 1
(b)
Plunger must not go above this point
Max. high position of plunger
Shim Plates
Location of Jack Plunger
to continue erection after connection of Upper
Back-stay at Panel Point No. 11. and disconnection
of Lower Back-stay at Panel Point No. 1.
(c)
For Normal Temp.
For Temperature 30° below Normal
Shim Plates
For Normal Temp.
For 30° below Normal
Location of Jack Plunger
to disconnect Upper Back-stay at
Panel Point No. 11.
(d)

FIG. 37.

caused excessive settlement of the comparatively soft ground on the Long Island side and interfered seriously with the closing operation. To relieve the foundation pressure, the removal of part of the earth fill from the counterweight box was started, therefore, immediately after the trusses had come into contact, and was kept up as fast as possible while the arch was being lowered into the self-supporting position. At the same time, levels were constantly taken to observe the settlements which took place. The average settlement observed was $2\frac{1}{2}$ in., which was not sufficient to cause any disturbance at the center. For emergency the eight 500-ton hydraulic jacks were ready to raise the counterweight, if this had become necessary.

The engineers and field force of the American Bridge Company, particularly C. G. E. Larsson, M. Am. Soc. C. E., Assistant Chief Engineer, who took personal charge, deserve full credit for the well-organized and careful manner in which this critical operation was carried out.

TABLE 5.—RECORD OF JACKING OPERATION FOR

Date.	Time.	Stage of closing operation.	Height of Shims. Observed. N. Truss. Inches.	Observed. S. Truss. Inches.	Calc. Inches.
Oct. 1, 1915.	9.30	Position of jacks unchanged since July 23, 1915. Started to raise jacks to loosen shim plates	$22\frac{9}{16}$	$22\frac{3}{8}$	....
	9.35	Shim plates loose. Started to lower jacks	24	$23\frac{7}{8}$	....
	10.05	Stopped lowering to bolt diagonals at 22 A	$21\frac{1}{4}$	$21\frac{1}{4}$	....
	10.23	Started lowering again	$21\frac{1}{4}$	$21\frac{1}{4}$	....
	10.30	Stopped lowering	$20\frac{5}{8}$	$20\frac{9}{16}$	....
	10.53	Raised south jack to match elev. of S. Truss of 22 A	$20\frac{3}{8}$	21	....
	11.00	Started lowering again	$20\frac{1}{8}$	$20\frac{1}{4}$	....
	11.15	Chords touching at 22 A, Temp. 56° Fahr.	$19\frac{7}{8}$	$19\frac{3}{4}$	$18\frac{15}{16}$
	11.28	Stopped lowering to bolt connections at 22 A.	$18\frac{3}{8}$	$18\frac{1}{4}$	$17\frac{3}{8}$
	1.05	Started lowering again and unloading counterweight	$18\frac{3}{8}$	$18\frac{1}{4}$	$17\frac{3}{8}$
	2.15	Finished lowering jacks, stays ¼ in. slack. Arch self-supporting, 3-hinged. Total settlement of counterweight, N = 3 in., S = 1⅞ in.	$4\frac{7}{8}$	$4\frac{3}{4}$	$4\frac{1}{16}$
Summary		Total lowering of jacks to make chords touch	$2\frac{11}{16}$	$2\frac{5}{8}$	$2\frac{5}{8}$*
		Additional lowering to release stays	15	15	$14\frac{3}{4}$

* Discrepancy between actual and calculated due to trusses being lower than calculated before

Transforming Trusses from Three-Hinged Into Two-Hinged Arches.—The transformation of the trusses from three-hinged into two-hinged arches required the connection of the top chord and one of the diagonals of the center panel of each truss at normal temperature (60° Fahr.). These members had been erected immediately after the closing of the arch, but had been left bolted at one end and free to move at the other.

The connection at the free end required drilling the rivet holes from the solid, and riveting. Although, during these operations, no elastic deformations of the arch trusses from loads took place, as the members were to be connected without initial stress at normal temperature, it was to be expected that, during the drilling and riveting, which work took several days, there would be changes in temperature. In order to prevent movements of the connection after the drilling of holes had once started, the Bridge Company resorted to an ingenious

CLOSING—HELL GATE ARCH—LONG ISLAND END.

Percentage of effective jacking completed.	Jack Pressure. Observed.	Jack Pressure. Calc.†	Opening Bet. Bottom Chords at Point 22. Observed.		Opening Bet. Bottom Chords at Point 22. Theoretical.	Elev. of Panel Point 22 L. I. Observed. N. Truss.	Elev. of Panel Point 22 L. I. Observed. S. Truss.	Elev. of Panel Point 22 L. I. Calc.	Condition of arch.
	Pounds per square inch.	Pounds per square inch.	Inches.		Inches.	Feet.	Feet.	Feet.	
			Actually observed at 56° Fahr.	Corrected to 60° Fahr.		Corrected to 60° Fahr. / Actually observed at 56° Fahr.		Theoretical for 60° Fahr.	
......	......	......	4 9/16	4 1/4	4 3/4*		....		Stage 12 A.
......	4 550	4 600			....				
......	3 950	4 600			....				
......	3 950	4 600			....				
......	3 900	4 600			....				
......	4 500	4 600	1 1/8	1 3/16	1 1/8				
......	3 850	4 600	1 1/4	1 5/16	7/8				
0	3 850	4 600	....	0	0	265.11 265.14	265.14 265.17	.... 265.19	Trusses in contact.
10	3 400	4 100	....	0	0				
10	3 400	4 100	....	0	0	265.18 265.21	265.21 265.24	.... 265.30	3-hinged.
100	0	0	....	0	0	Relative Elevations.			
						— 1 1/16 in.	— 3/4 in.	0.0	
......	....	....	....	....	....	....	....	....	
......	....	....	....	....	....	....	....	....	

position. † No correction for friction of jacks. ‡ Calculated for opening observed
jacking.

device to hold the ends in place. A rod, 8 in. in diameter and 16 ft. long, of sufficient strength to resist any possible tension or compression from changes in temperature, was introduced into each top chord, extending across the open joint of the chord at Panel Point 23, Wards Island side. Each end of the rod was secured with double nuts to a diaphragm riveted to the chord (Fig. 38).

On a favorable day, when the temperature was practically normal and uniform over the whole structure, the nuts on these rods were tightened, with a slight initial stress in the rods. From this moment all stresses were taken by the rods, thus relieving the ends of the chords and permitting the drilling and riveting without disturbance. The rods were left in place, and therefore partook in resisting the chord stresses.

Field Riveting, Bolting, and Drifting.—A total of 333 960 field rivets, or about 17 per ton of steelwork, had to be driven in the Hell Gate Bridge, not including those driven in some connections of the back-stays. About two-thirds of this number, or 202 404, are 1¼ in. in diameter and have grips up to 9⅜ in. Their shape has been described under "Workmanship and Fabrication." The connections of the back-stay trusses were in general made with 80% of drift-pins and 20% of bolts.

All truss connections were made temporarily with bolts and drift-pins. From 25 to 60% of the rivet holes were filled with drift-pins, or a number sufficient to carry the entire erection stress. About 50% of the holes were filled with bolts to tie the different parts firmly together. The bolts were not supposed to transmit any stress. The riveting of the connection of the web members and of the six end panels of the top chord was permitted to proceed by gradually replacing the pins and bolts with rivets after the erection had proceeded at least three panels ahead. The riveting of the bottom chord connections and of the middle panels of the top chord, however, was deferred until the arch had become self-supporting and the bottom chord carried the greater part of the dead load. The object was to allow the joints of these compression chords to come into full and forcible contact, and transmit a greater unit stress than the splicing material after the riveting had been completed. As the bottom chords had been planed at one end to a three-plane face, as explained previously, the two outer

thirds of each joint formed wedge-shaped openings when the member was put in place. As had been anticipated, the openings gradually closed as the stress increased, and particularly when the drift-pins were replaced by rivets. It can be assumed, therefore, that the bearing stress across the joint from dead load increases from nearly zero at the edges to a maximum over the middle third of the joint, and that dangerous edge pressures are thus avoided. Part of the dead-load stress, of course, remained in the splice material, as the gradual replacing of the drift-pins by rivets did not entirely relieve the splice material from stress. The additional stresses from live load and other forces are shared proportionally by the full joint and the splice material.

All field riveting was done by pneumatic hammers with pneumatic bucking-up as described elsewhere. Air was delivered with a pressure at the tool of about 100 lb. per sq. in., from two compressors, one on each side of the river. From twelve to sixteen riveting gangs were employed on the bridge. The average daily output for rivets 1⅛ in. in diameter was 165 per gang and the maximum daily output of one gang was 338.

Storing, Shipping, Handling, and Hoisting Material.—Material from the shops was delivered by rail and stored at the Pennsylvania Railroad freight yards at Greenville, N. J. Special box cars were required for the transportation of the heavy, deep chord members (Fig. 35). From Greenville the material was transhipped on car-floats up the East River to the bridge site as needed during the erection.

On both shores on the river docks had been built for unloading the materials, each having been provided with a 65-ton double stiffleg derrick. These docks were also used for delivering materials for the masonry. The members for the three end panels of the arch were raised from the ground or docks by the arch traveler (Fig. 36), those for the other panels were floated on a barge to a position under the arch traveler, and raised by the latter directly from the barge (Fig. 38). To facilitate and expedite the lifting and setting into place, a special hitch was temporarily attached to each heavy member above its center of gravity. This was placed so that the member would hang in the same relative position as it was to occupy in the structure.

Travelers.—The four powerful creeper travelers, one pair on the back-stays and one pair on the arch proper, had been specially designed

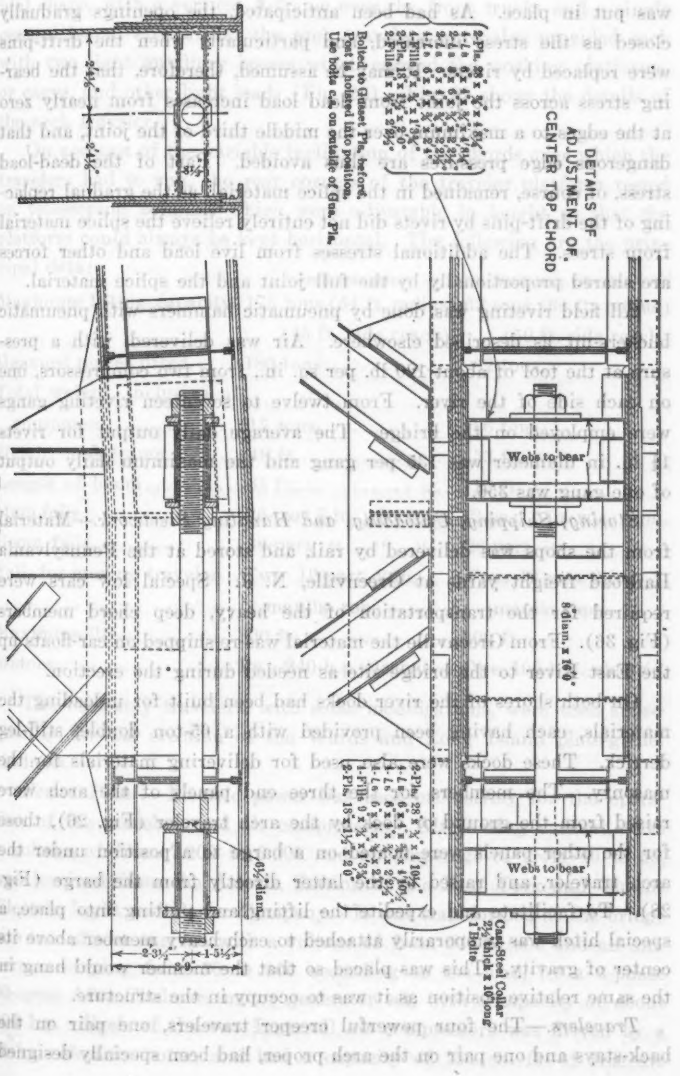


FIG. 38.

thirds of each joint formed wedge-shaped openings when the member was put in place. As had been anticipated, the openings gradually closed as the stress increased, and particularly when the drift-pins were replaced by rivets. It may be assumed, therefore, that the bearing stress across the joint from dead load increases from nearly zero at the edges to a maximum over the middle third of the joint, and that dangerous edge pressures are thus avoided. Part of the dead-load stress, of course, remained in the splice material, as the gradual replacing of the drift-pins by rivets did not entirely relieve the splice material from stress. The additional stresses from live load and other forces are shared proportionally by the full joint and the splice material.

All field riveting was done by pneumatic hammers with pneumatic buckers-up, as described elsewhere. Air was delivered, with a pressure at the tool of about 120 lb. per sq. in., from two compressors, one on each side of the river. From twelve to seventeen riveting gangs were employed on the bridge. The average daily output for rivets 1¼ in. in diameter was 135 per gang and the maximum daily output of one gang was 356.

Storing, Shipping, Unloading, and Handling Steelwork.—Material from the shops was delivered by rail, and stored at the Pennsylvania Railroad freight yards at Greenville, N. J. Special low cars were required for the transportation of the heavy, deep chord members (Fig. 36). From Greenville the material was re-shipped on car-floats up the East River to the bridge site as needed during the erection.

On both shores of the river docks had been built for unloading the materials, each having been provided with a 65-ton double, stiff-leg derrick. These docks were also used for delivering materials for the masonry. The members for the three end panels of the arch were raised from the ground or dock by the arch traveler (Fig. 26), those for the other panels were floated on a barge to a position under the arch traveler, and raised by the latter directly from the barge (Fig. 28). To facilitate and expedite the lifting and putting into place, a special hitch was temporarily attached to each heavy member above its center of gravity. This was placed so that the member would hang in the same relative position as it was to occupy in the structure.

Travelers.—The four powerful creeper travelers, one pair on the back-stays and one pair on the arch proper, had been specially designed

and built for the erection of the arch bridge. They were of the pyramidal shape, with a vertical A-frame over the front trucks and a single central boom, except that the arch travelers were also provided each with two light auxiliary booms which carried the working platforms, or cages, and other light loads (Fig. 26). Fig. 39 shows the details of the arch traveler.

On account of the variable inclination of the chords over which the travelers had to run, the rear corners of the traveler platform rested on telescoping columns which were adjustable in height so that the platform could always be kept horizontal. The following are the principal data:

	Arch traveler.	Back-stay traveler.
Maximum lifting capacity.	175 tons (54 ft. radius) (45 ft. side reach).	62 tons (64 ft. radius) (50 ft. side reach)
Heaviest pieces lifted	180 tons	62 tons.
Total weight, inclusive of equipment	315 tons	166 tons.
Height of A-frame	48 ft.	53 ft.
Length of boom	65 ft.	112 ft.
Main falls	26-part $\frac{7}{8}$-in. wire rope.	12-part $\frac{3}{4}$-in. wire rope.
Boom falls	36-part " " "	26-part " " "
Falls for moving traveler	Two 12-part 2-in. manila rope	Four 12-part 2-in. manila rope.
Maximum lift	300 ft.	300 ft.
Motors	Two 240-h.p. electric	One 100-h.p. electric.

The back-stay travelers, with some modifications, were used subsequently for the erection of the Wards and Long Island plate-girder viaducts.

Power Plant.—Electric power was used exclusively for the operation of the travelers and air compressors. Alternating current, with a voltage of from 7 200 to 8 200 and an amperage of from 40 to 55, was received from the Astoria Station of the New York and Queens Electric Light and Power Company. It was transformed at the bridge site into direct current with a voltage of from 550 to 600 and an amperage varying from 50 to 900 according to the load, by a 3-phase 60-cycle Allis Chalmers motor generator set, with a capacity of about 500 h.p. Each of the two Ingersoll air compressors was driven by a 75-h.p. electric motor, and had a capacity of 225 cu. ft. of free air

compressed per minute to a pressure of at least 135 lb. per sq. in. at the cylinder.

Progress.—The following are the principal dates in connection with the erection of the Arch Bridge (see also Plate XXXIII):

	Long Island side.	Wards Island side.
Erection of back-stays started........	September, 1914	January, 1915
Erection of arch proper started.....	January, 1915	May, 1915
First six panels erected...........	June, 1915	August, 1915
Arch closed.......................	October 1st, 1915	
Suspenders and floor system erected..	January, 1916	
Back-stays dismantled...............	April, 1916	
Riveting completed.................	September, 1916	

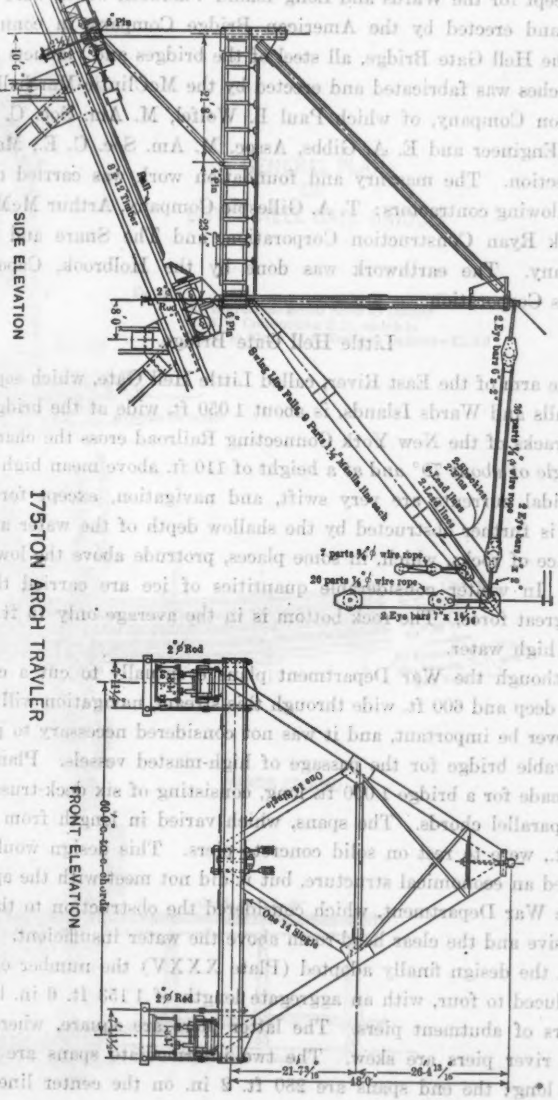
Contractors' Organization.—The work of the American Bridge Company was under the general charge of C. W. Bryan, M. Am. Soc. C. E., Chief Engineer, and the immediate charge of Mr. C. G. E. Larsson, Assistant Chief Engineer. Mr. J. B. Gemberling, Division Erection Manager, had charge of the erection. The contractor's field organization consisted of a general foreman, three assistant engineers, one foreman, and an average daily force of 140 specially picked men. The maximum force at any one time was 270 men. There were two separate erection gangs, one on each side of the river.

Unusual precautions were taken for the safety of the workmen. Only 5 men lost their lives, mostly due to their own carelessness. Hand ropes and temporary wooden railings were provided on many members, and special steel cages with comfortable platforms were hung from the travelers or from the steelwork for the convenience of the men working on the truss connections (Fig. 33).

12.—Approaches.

The bridge and viaduct approaches to the Hell Gate Bridge consist of 2 735 lin. ft. of steel truss bridges, 10 818 lin. ft. of plate-girder viaducts, and 3 228 lin. ft. of embankment between reinforced concrete retaining walls, with arches over the streets. Except for the bascule spans of the Bronx Kill Bridge, which have open tie flooring, all bridges and viaducts carry ballasted tracks on concrete slabs, weighing 3 500 lb. per lin. ft. of single track. The live load, quality of steel, and permissible unit stresses are given under "Material" and "Loads and Unit Stresses."

Except for the Wards and Long Island Viaducts, which were fabricated and erected by the American Bridge Company in connection with the Hell Gate Bridge, all steel for the bridges and sections of the approaches was fabricated and erected by the McClintic-Marshall Construction Company, of which Paul L. Wolfel, M. Am. Soc. C. E., is Chief Engineer and E. A. Gibbs, Assoc. M. Am. Soc. C. E., Manager of Erection. The masonry and foundation work was carried out by the following contractors: T. A. Gillespie Company, Arthur McMullen, Patrick Ryan Construction Corporation, and The Snare and Triest Company. The earthwork was done by the Holbrook, Cabot and Rollins Corporation.



175-TON ARCH TRAVLER
FIG. 39.

Little Hell Gate Bridge.

The arm of the East River, called Little Hell Gate, which separates Randalls and Wards Islands, is about 1 050 ft. wide at the bridge site. The tracks of the New York Connecting Railroad cross the channel at an angle of about 75° and at a height of 110 ft. above mean high water. The tidal currents are very swift, and navigation, except for small craft, is further obstructed by the shallow depth of the water and the presence of rocks which, in some places, protrude above the low-water level. In winter considerable quantities of ice are carried through with great force. The rock bottom is in the average only 5 ft. below mean high water.

Although the War Department plans eventually to cut a channel 24 ft. deep and 600 ft. wide through Little Hell Gate, navigation will probably never be important, and it was not considered necessary to provide a movable bridge for the passage of high-masted vessels. Plans were first made for a bridge 1 050 ft. long, consisting of six deck-truss spans with parallel chords. The spans, which varied in length from 145 to 205 ft., were to rest on solid concrete piers. This design would have secured an economical structure, but it did not meet with the approval of the War Department, which considered the obstruction to the river excessive and the clear headroom above the water insufficient.

In the design finally adopted (Plate XXXV) the number of spans is reduced to four, with an aggregate length of 1 153 ft. 6 in. between centers of abutment piers. The latter are square, whereas the three river piers are skew. The two intermediate spans are 290 ft. 6 in. long; the end spans are 280 ft. 2 in. on the center line of the

Except for the Wards and Long Island Viaducts, which were fabricated and erected by the American Bridge Company in connection with the Hell Gate Bridge, all steel of the bridges and viaducts of the approaches was fabricated and erected by the McClintic-Marshall Construction Company, of which Paul L. Wolfel, M. Am. Soc. C. E., is Chief Engineer and E. A. Gibbs, Assoc. M. Am. Soc. C. E., Manager of Erection. The masonry and foundation work was carried out by the following contractors: T. A. Gillespie Company, Arthur McMullen, Patrick Ryan Construction Corporation, and The Snare and Triest Company. The earthwork was done by the Holbrook, Cabot and Rollins Corporation.

Little Hell Gate Bridge.

The arm of the East River, called Little Hell Gate, which separates Randalls and Wards Islands, is about 1 050 ft. wide at the bridge site. The tracks of the New York Connecting Railroad cross the channel at an angle of about 70° and at a height of 110 ft. above mean high water. The tidal currents are very swift, and navigation, except for small craft, is further obstructed by the shallow depth of the water and the presence of rocks, which, in some places, protrude above the low-water level. In winter considerable quantities of ice are carried through with great force. The rock bottom is in the average only 15 ft. below mean high water.

Although the War Department plans eventually to cut a channel 24 ft. deep and 600 ft. wide through this stream, navigation will probably never be important, and it was not considered necessary to provide a movable bridge for the passage of high-masted vessels. Plans were first made for a bridge 1 050 ft. long, consisting of six deck-truss spans with parallel chords. The spans, which varied in length from 145 to 205 ft., were to rest on solid concrete piers. This design would have secured an economical structure, but it did not meet with the approval of the War Department, which considered the obstruction to the river excessive and the clear head-room above the water insufficient.

In the design finally adopted (Plate XXXV) the number of spans is reduced to four, with an aggregate length of 1 153 ft. 6 in. between centers of abutment piers. The latter piers are square, whereas the three river piers are skew. The two intermediate spans are 296 ft. 6 in. long; the end spans are 280 ft. 2 in. on the center line of the

PLATE XXXV.
TRANS. AM. SOC. CIV. ENGRS.
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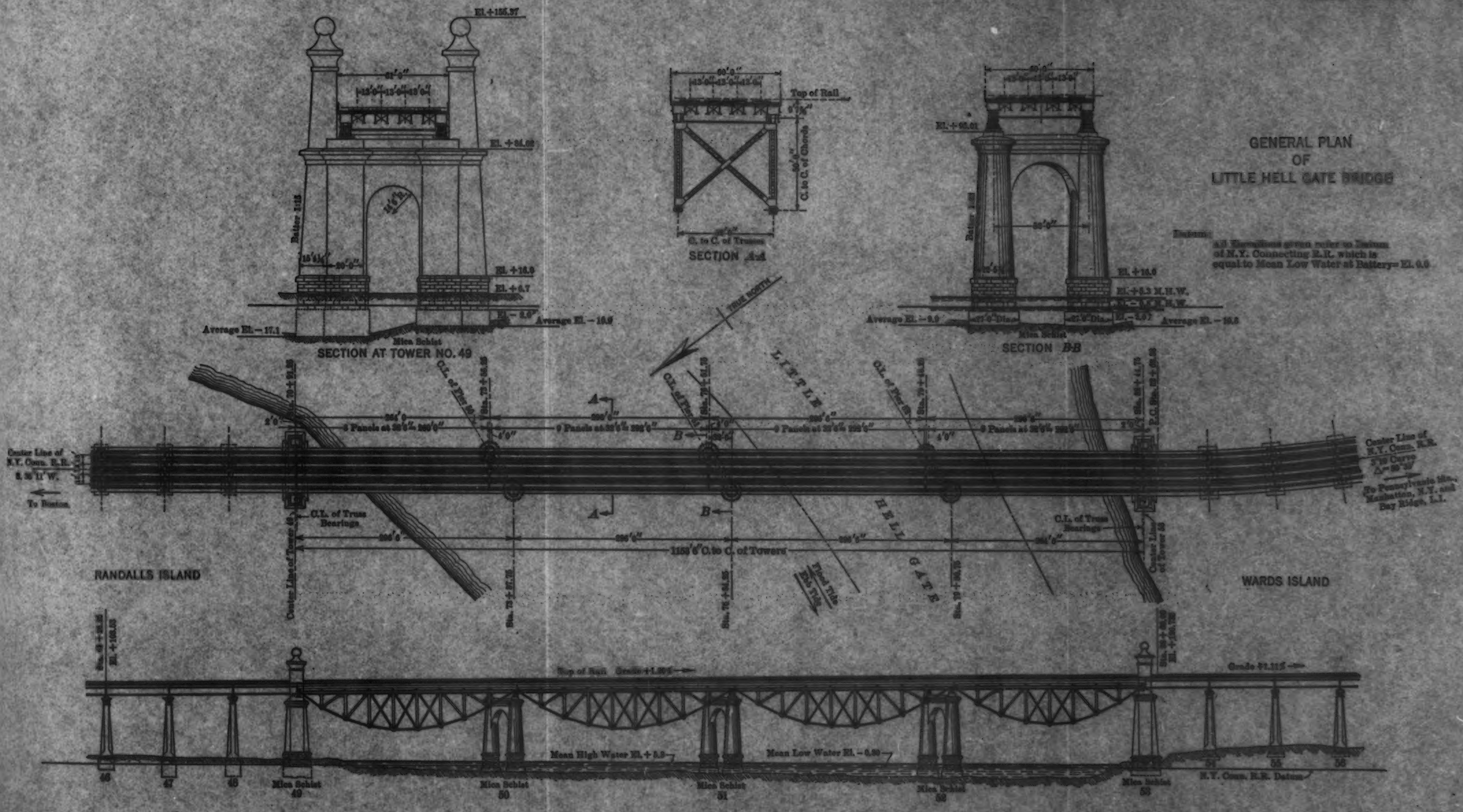
AMMANN ON
HELL GATE BRIDGE.



bridge, or 296 ft. 6 in. on the center line of the longer truss, so that only two different sizes of trusses were required. The deck trusses, two for each span, were made of the bowstring type so as to give additional clear height near the piers for higher vessels which might occasionally pass underneath. The river piers were reduced in size, each to two cylindrical shafts 25 ft. in diameter at the water line, so as to offer the least resistance to the flow of water and ice. A through-truss bridge would have given more clearance, but would have been considerably more expensive on account of the greater width, and would not have harmonized in appearance with the approach viaducts as well as the adopted type. The architecture of the piers and abutments is similar to that of the viaduct piers. The slender, slightly battered river piers, in combination with the graceful outline of the trusses, give the bridge a very pleasing appearance (Fig. 40). The ends of this important structure are befittingly marked by towers extending above the track floor at each abutment.

All foundations are on solid rock, and were placed by open cofferdams. The piers and abutments are of concrete, except that between Elevation 10.7 ft. above mean high water and 1.2 ft. below mean low water the concrete is protected against disintegration by granite facing.

The two trusses are 52 ft. apart on centers, and have a greatest depth of 50 ft. at the center. The bottom chord has a parabolic shape, and is made up of from ten to thirteen lines of structural-steel eye-bars, 16 in. wide, up to 2¼ in. thick, and more than 38 ft. long, connected by 16-in. pins. All other members are riveted. Eye-bars were selected in this case, as they effected a considerable saving over riveted chords. The type of truss is economical, as the chords are of nearly uniform section and the web members are very light. The secondary stresses in the end panels are comparatively large, but those from dead load have been practically eliminated by making the joints and angles between members in accordance with the geometric form of truss and swinging the spans clear of falsework before the connections were riveted. The trusses are connected by a rigid top lateral system and also by stiff sway-frames at each vertical post. No lateral system is provided along the bottom chord, as it was considered preferable to have the wind force, which acts along this chord, transmitted to the lateral system between the stiffer top chords.

The two end spans have fixed bearings over the abutments; all bearings over the piers are movable, an expansion joint being provided over the center pier. With this arrangement the piers could be made considerably narrower, as they do not have to resist the longitudinal force from braking. To avoid the transmission of any longitudinal force to the piers through friction, the bearings are provided with cast-steel rockers of the unusual height of 24 in. The relative movement of the two bearings at the center pier is as much as ± 6 in.

The four spans were erected successively on unusually heavy timber falsework (Fig. 41). A total of 360 000 ft. b.m. of timber was required for each span. Each panel point was supported by a double bent strongly braced transversely and longitudinally. The piles had to be braced above and below water level on account of their small penetration and the strong current of the river. The steel was handled with a steel gantry traveler, which weighed 275 tons, inclusive of equipment. Erection was started in February, and completed in November, 1915.

The bridge contains 25 700 cu. yd. of masonry and 11 250 tons of steelwork. The cost of construction, inclusive of tracks, was approximately $990 000.

Bronx Kill Bridge.

The tracks of the New York Connecting Railroad cross Bronx Kill, which separates Randalls Island from the main land, at an angle of about 72° with the channel and at a height of 75 ft. above mean high water. Bronx Kill is here about 600 ft. wide between shore lines, and 320 ft. between bulkhead lines as established by the War Department. Under present conditions, it is navigable only for small boats. The river bottom consists of various strata of river mud, sand, gravel, and clay overlying the gneiss bed-rock, the surface of which falls from both shores to a maximum depth of about 110 ft. below mean high water under the north abutment. According to plans of the War Department, Bronx Kill is to be improved by dredging a channel 24 ft. deep and 480 ft. wide, principally for the purpose of decreasing the excessive tidal currents in the East River, and incidentally to provide a direct passageway for deep-draft vessels between the Harlem River and the upper East River. For this reason the War Department required a movable bridge having two clear openings, of at least 120 ft. each, at right angles to the channel.

FIG. 40.—LITTLE HELL GATE BRIDGE.

FIG. 41.—ERECTION OF LITTLE HELL GATE BRIDGE.

The two end spans were built [illegible] with the abutments; all bearing over the piers are movable, an expansion point being provided over the [illegible] span. [illegible]

FIG. 40.—LITTLE HELL GATE BRIDGE.

The bridge contains [illegible] cu. yd. of masonry and 11,250 tons of steelwork. The cost of construction, exclusive of tracks, was approximately [illegible].

Bronx Kill Bridge

The tracks of the New York Connecting Railroad cross Bronx Kill, which separates Randall's Island from the main land, at an angle of about [illegible] with the channel and at a height of [illegible] ft. above mean high water. [illegible]

FIG. 41.—ERECTION OF LITTLE HELL GATE BRIDGE.

[illegible] between the Harlem River and the upper East River. For this reason the War Department required a movable bridge having two clear openings of at least 125 ft. each, at right angles to the channel.

PLATE XXXVI.
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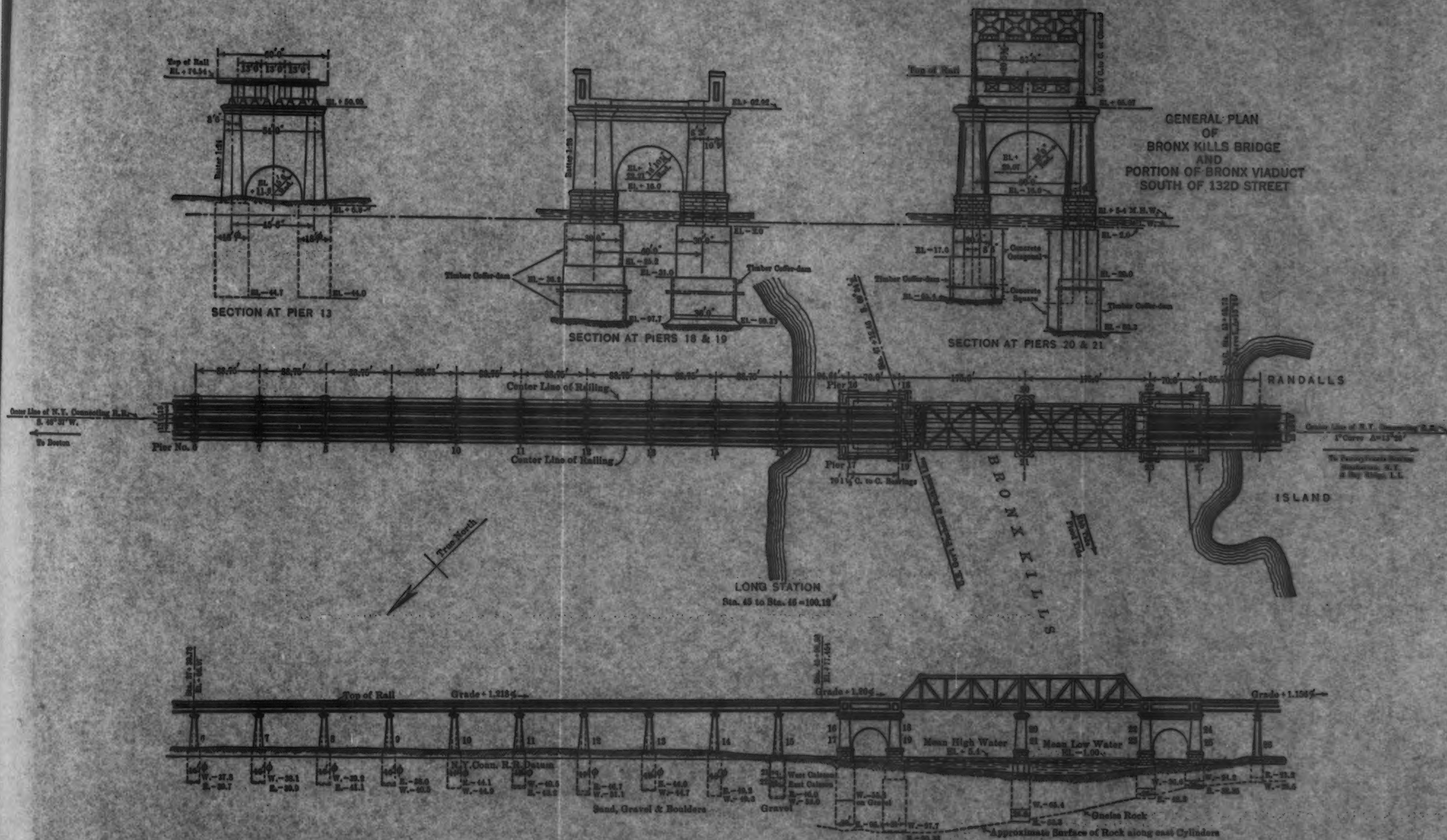
AMMANN ON
HELL GATE BRIDGE.



The available height of 75 ft. from top of rail to mean high water would have been sufficient for a deck bridge, which would have been more economical than a through structure. The latter type was adopted, however, in order to provide sufficient clear height for all ordinary navigation to pass underneath, and thus restrict the frequency of opening the bridge and the consequent delay to railroad traffic. This consideration was important in view of the prospective heavy traffic, and on account of the heavy grade, on which it is difficult to start trains. Comparative designs were made of various types of movable bridges, including the horizontal draw, the Scherzer rolling lift, and the Strauss trunnion bascule.

The horizontal draw-bridge would have necessitated a very wide pier which would have obstructed the channel excessively and would have required very costly foundations. The Strauss bascule was finally adopted because it was found to be cheapest in first cost, that is, without operating machinery, counterweights, etc., which may not have to be put in for many years. The only parts of the moving mechanism which had to be provided are the trunnions and their bearings, and certain connections for the future attachment of the counterweight trusses.

The bridge as built (Plate XXXVI) has two leaves, each 175 ft. long between centers of trunnion bearings and middle pier. Each leaf acts as a single span when closed. All four tracks are carried between the two trusses, which are 60 ft. apart on centers and 45 ft. deep. The trusses are fully riveted.

Each abutment consists of four rectangular piers, connected in pairs longitudinally and transversely by arched concrete walls, and thus forming a rectangular enclosure which will contain the counterweights and hide them from view when the bridge is closed. The two transverse walls support a 70-ft. plate-girder span which now carries the tracks and, ultimately, will carry the operating machinery. The intermediate pier consists of two circular shafts joined by a light concrete arch. The abutments and pier are architecturally treated to conform in appearance to the adjoining viaduct piers (Fig. 42).

All piers were sunk by the pneumatic process, with timber caissons, to depths of from 30 to 105 ft. below mean high water. At the water level the concrete is protected with granite facing in a manner similar to that adopted for the piers of the Little Hell Gate Bridge. The

steel trunnions, around which the trusses will turn, are of exceptional size, being 30 in. in diameter and 10½ ft. long. The tracks rest on open tie flooring securely fastened to the steel floor system.

The bridge was erected on heavy timber falsework, one span at a time, 256 000 ft. b.m. of lumber having been used in each span. The members were handled with the 50-ton derrick car and a locomotive crane, and these were used for the erection of the adjoining viaducts. Erection was started in August, 1914, and completed in March, 1915. The principal quantities in this bridge are:

Masonry	28 300 cu. yd.
Steelwork:	
Two 70-ft. plate-girder spans...........	470 tons
Two 175-ft. bascule spans (exclusive of machinery and counterweights)....	3 105 tons
Total	3 575 tons

Counterweights, operating machinery, and their supports are estimated to require about 750 tons of steel and 1 300 cu. yd. of concrete filling. The cost, exclusive of counterweights and operating machinery, but inclusive of tracks, was approximately $800 000.

Bronx Viaduct North of 132d Street.

General Conditions.—The design and construction of the portion of The New York Connecting Railroad designated as "Bronx Viaduct north of 132d Street", extending from 132d Street, where the New Haven and the Connecting Roads converge, to the crossing over the New York and Harlem Railroad, about 380 ft. north of 141st Street (Plate XXXVII), presented considerable difficulties on account of the unfavorable character of the soil and the proximity of the four New Haven tracks, with a siding along the eastern side of the New York Connecting Railroad from 132d Street to the De La Vergne Machine Works at 137th Street. The two easterly New Haven tracks were originally located alongside the two westerly tracks and, therefore, had to be shifted eastward, to make place for the construction of the two westerly New York Connecting tracks. This had to be done without interruption of traffic on the New Haven tracks. Borings along this entire section indicated a top layer of gravel, coarse sand,

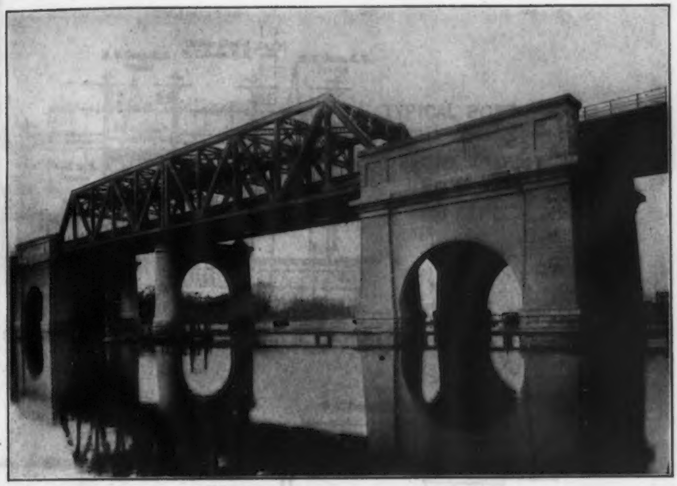


Fig. 42.—Bronx Kill Bridge.

Fig. 43.—Concrete Arch over Debevoise (Second) Avenue, Long Island City.

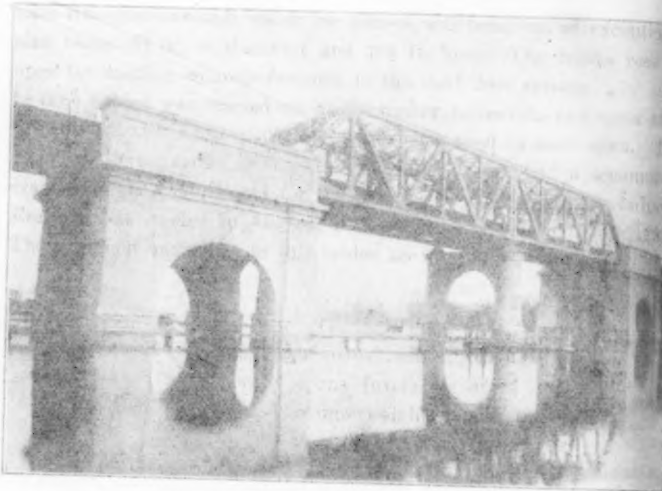


FIG. 42.—BRONX KILL BRIDGE.

FIG. 43.—CONCRETE ARCH OVER DEPRESSED (SECOND) AVENUE, LONG ISLAND CITY.

PLATE XXXVII.
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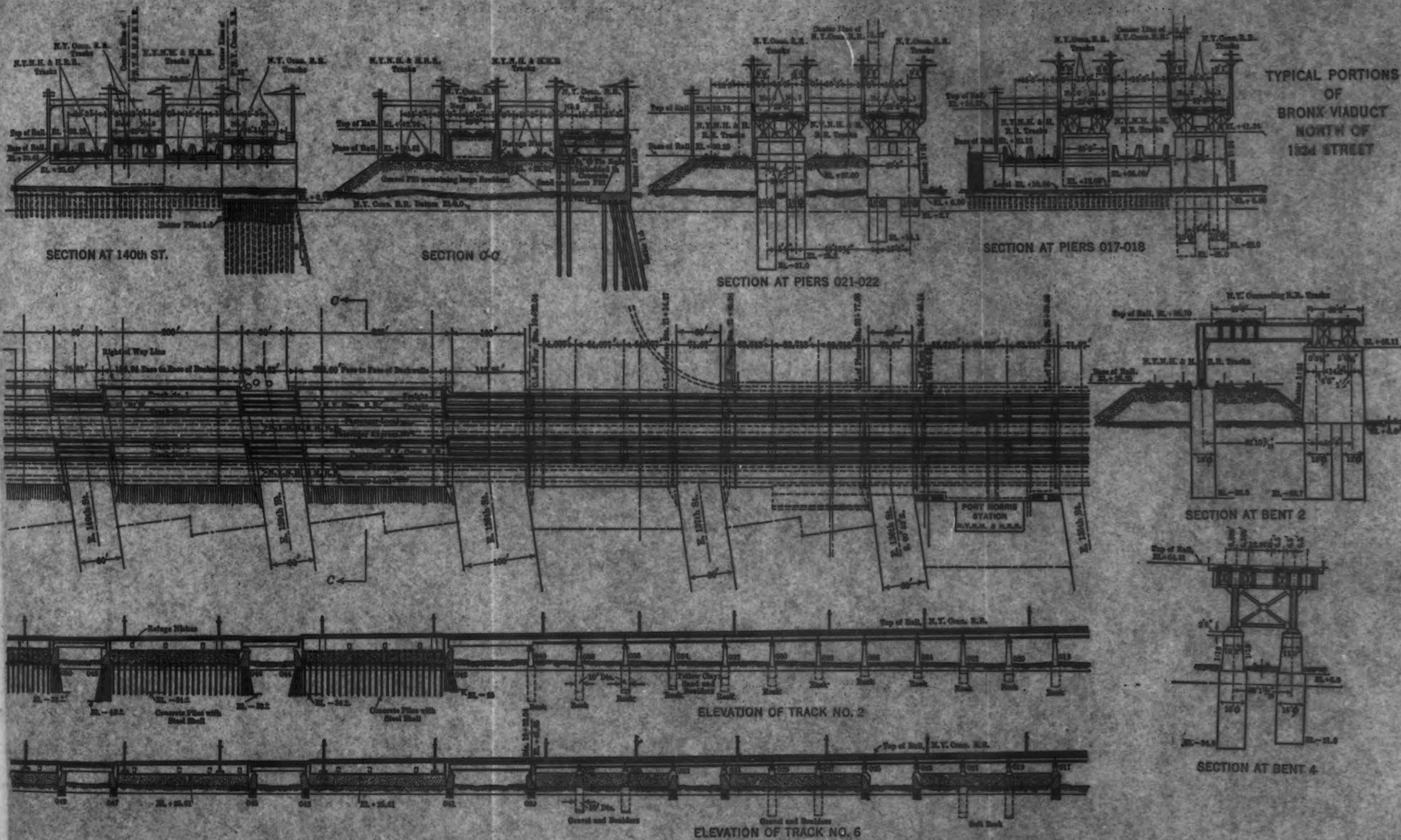

and ashes, from 6 to 12 ft. deep, below this a soft stratum of silt and mud from 12 to 25 ft. deep, resting on a harder stratum of sand and gravel. Only under the easterly tracks near 136th Street does the rock rise to a few feet below the surface.

In 1906, or about 6 years previous to the construction of the Connecting Railroad, the New Haven Railroad had raised its four tracks on an embankment, 65 ft. wide at the top and about 20 ft. above the original ground surface. This embankment consists of cinders, earth, gravel, and a great quantity of large boulders; the latter have gradually settled several feet into the original soft ground and have caused it to heave on the sides. Subsequently, this embankment had to be widened to 95 ft. at the top, when the two easterly tracks were shifted to their present location. The vicinity of this portion of the line is built up mainly with industrial buildings, for which reason no particular consideration had to be given to the architectural treatment of the structure.

Embankment Portion.—On the portion of the Bronx Viaduct between the crossing over the New York and Harlem Railroad and 138th Street, it was found most economical to place the four New York Connecting tracks on fill between reinforced concrete retaining walls, the streets being crossed on deck plate-girder spans. The two retaining walls which enclose the fill under the two westerly tracks—Nos. 5 and 6—rest directly on the New Haven embankment. They are connected by transverse walls, about 20 ft. apart, and thus form, in each block between streets, a monolithic cellular box without bottom. Its maximum height is 19 ft. This box, with the fill between, forms a compact unit which exerts a practically uniform pressure of not more than 1½ tons per sq. ft. on the embankment beneath. Settlements of the retaining walls were expected, but they can have no serious consequences. Since their completion in 1914 a maximum settlement of 8 in. has been observed, without any sign of cracks in the walls. The abutments of the street crossings are independent of the retaining walls, and rest on timber piles. This plan was simple, and proved to be more economical than any other that could have been used. Piles under the retaining walls would have been impracticable on account of the boulder formation of the New Haven embankment, and the expense of any other foundation reaching down to hard soil would have been excessive.

The conditions under the two easterly New York Connecting tracks —Nos. 1 and 2—were different. There, no embankment existed on top of the original surface. The easterly retaining wall under Track No. 1 was carried down to about 7 ft. below the ground surface or 2 ft. below the probable future ground-water level, and was placed on timber piles, from 35 to 45 ft. long, driven to refusal into the hard strata of sand and gravel. The maximum pressure per pile is approximately 20 tons. The westerly retaining wall under Track No. 2 was to have been placed on the new fill between the original New Haven embankment and the retaining wall under Track No. 1. After this fill, which consists largely of sand and loam without boulders, had been placed, it settled considerably, and it was deemed inadvisable to place the wall thereon. Therefore, hollow steel piles, 12 in. in diameter and about 60 ft. long, with cast-steel shoes, were driven through the fill into the hard soil. After being cut off at the proper elevation, the piles were filled with concrete. The wall was then placed on top of these piles. The maximum pressure per pile is 45 tons. To prevent any lateral motion of this wall, due to greater earth pressure on the east side, it is connected to the wall under Track No. 1 with steel tie-rods encased in concrete.

Both retaining walls under Tracks Nos. 1 and 2 are of L-shape and are stiffened by vertical buttresses, 8 ft. apart. They are heavily reinforced with steel rods. This portion of the Bronx Viaduct, about 1 173 ft. long, contains approximately 16 000 cu. yd. of earth fill, 135 000 lin. ft. of timber piles, 8 800 lin. ft. of steel piles, 12 000 cu. yd. of concrete masonry, and 855 tons of structural steelwork. The cost of construction, inclusive of tracks but exclusive of electrification, was about $230 per lin. ft. of 4-track road.

Viaduct Portion.—The embankment type of construction, as previously described, was not practicable south of 138th Street, because, on account of the greater height, the foundation pressures would have become excessive. The plate-girder type of viaduct, with concrete piers, was adopted as being most suitable (Plate XXXVII). The spans are generally about 64 ft. long, except over the streets, where they had to be from 72 to 112 ft. The piers are of plain rectangular shape, with simple square copings.

To be safe, the foundations for the piers had to go down through the silt to the hard strata of sand and gravel or rock. Piles would

have been the least expensive foundation, but, in view of the considerable depth of soft, yielding material, they would not have afforded sufficient stability against lateral forces or lateral motion which might be caused by any disturbance of the soft ground in the future construction of adjacent foundations. Furthermore, piles could not have been driven through the boulder formation of the New Haven embankment under the two westerly tracks. Therefore, the more expensive, but securer type of foundation, consisting of two cylindrical concrete caissons under each pier, was adopted. The cylinders are from 10 to 15 ft. in diameter and were sunk partly by open dredging, and partly by the pneumatic process, to depths of from 12 to 55 ft. below the surface. The greatest pressure on the foundations is approximately 8 tons per sq. ft.

A deviation from the foregoing type of viaduct was necessitated at 133d and 132d Streets, where the New Haven and New York Connecting Railroads diverge. On account of insufficient space for piers, it was necessary to frame the longitudinal girders of the two westerly tracks into cross-girders supported by steel columns. The plate-girder viaduct portion, 1 736 ft. long, contains 14 800 cu. yd. of concrete masonry and 7 080 tons of steel work. The cost of construction was about $365 per lin. ft. of 4-track road.

Bronx Viaduct South of 132d Street, Randalls, Wards, and Long Island Viaducts.

The general conditions which affected the design of the viaducts in the Bronx section south of 132d Street, on Randalls and Wards Islands, and on Long Island north of Lawrence Street, are similar, with the exception of the character of the soil. The fact that these four sections are prominently exposed to view called for a uniform and pleasing appearance, in harmony with the monumental character of the arch bridge which they flank on both sides. The type of viaduct adopted consists of deck plate-girder spans resting on arched concrete piers. The span length varies from 72 to 94 ft., except for a single span over Van Alst Avenue in Long Island, which is 130 ft. The lengths chosen are the most economical, except on the Long Island viaduct, where they were largely determined by the location of the many streets which had to be crossed.

On the Bronx Viaduct the piers rest on cylinder caissons, from 15 to 18 ft. in diameter, one under each leg of the pier. These cylinders were sunk partly by open dredging, and partly under air pressure, to depths varying from 45 to 60 ft. below the surface, through soft silt and mud to a hard stratum of sand and gravel or solid rock (Plate XXXVII). On all other viaduct sections the piers have ordinary footings built in open excavation. On the Long Island Viaduct, where the soil has various formations of sand, gravel, boulders, loam, and clay, partly saturated with water which is likely to be drained off, all the footings had to be spread considerably so as to decrease the bearing pressure to the safe value of about 2½ tons per sq. ft., and had to go to depths as great as 34 ft. below the surface.

On Wards and Randalls Islands the piers have comparatively shallow foundations, either on solid rock, or on hard strata of sand and gravel, or hardpan, which permitted pressures of 4 tons per sq. ft. and greater.

The proximity of the railroad to residential districts and to public parks made it desirable to restrict the noise from trains. Embankments between retaining walls were out of the question on account of the great height. Had soil conditions been favorable throughout, a viaduct consisting of a series of solid concrete arches would have been most suitable, as regards appearance and restriction of noise. This type is expensive in first cost, but is more durable, and less costly to maintain, than a plate-girder viaduct, the steel superstructure of which may have to be replaced or strengthened at some future date, and will require frequent painting. Concrete arches, however, require solid, unyielding foundations, in order to prevent dangerous and unsightly cracks, and such foundations could not be had on every section at reasonable cost.

Comparison of Three Designs of Plate-Girder Viaducts.

Fig. 44 shows typical portions of two preliminary designs for the plate-girder viaducts, and the design adopted. Fig. 44(*a*) represents the typical American trestle viaduct with alternate tower spans, 40 ft. long, and intermediate spans of 80 ft. This type, the advantages of which were cheapness and rapidity of erection, is now gradually being displaced by types of greater rigidity and durability, and better appearance. It would have been inappropriate and inadequate for the approaches to the Hell Gate Bridge. Fig. 44(*b*) represents the

design made by Mr. Lindenthal in 1908. It consists of plate girders of nearly uniform span length of from 70 to 80 ft., resting on steel rocker bents. To resist the longitudinal forces from braking and traction, solid masonry piers (stability piers) were to be provided at about every tenth span. This design is superior in general appearance to the trestle design. The stability piers convey the impression of rigidity, and give opportunity for architectural treatment. The arch form selected for the steel rocker bents, although somewhat more expensive, is more pleasing than the ordinary two-column bent with single intersecting diagonals. This type of viaduct is also stiffer in the longitudinal direction at least, than the trestle type. Fig. 44 (c) represents the type finally adopted, with concrete piers. It is superior to the other two types in appearance, rigidity and durability, and is less costly to maintain.

A comparison of estimated costs, with the prices prevailing at the time the design was made, showed that, for an average height of viaduct of 100 ft. on tangent, the steel trestle design would have been about 20% cheaper, and the design with steel rocker bents about 10% cheaper, than the adopted design. On a 3° curve the saving in first cost would have been only 15% and 8%, respectively, as the centrifugal force of the trains requires additional material in the steel bents and towers but not in the masonry piers. For heights of viaducts of less than 100 ft., the differences in cost are correspondingly less. With the high prices of steel prevailing at present, there would be little, if any, saving in favor of the steel trestle type.

The arched concrete piers mark a radical departure from the ordinary solid square concrete piers with plain surface and simple square coping. The rectangular body of the pier proper is only 6 ft. thick from the coping down, but is reinforced by four buttresses which have a batter of 1 to 15. These piers convey the impression of elegance and yet of great rigidity. The appearance is enhanced by the massive and architecturally elaborate capping of cornices and mouldings.

The concrete is made of 1 part Portland cement, 2 parts sand, and 4 parts gravel or broken stone, and is reinforced with steel rods, vertically and horizontally, against shrinkage and temperature cracks.

The plate-girder spans, typical details of which are shown in Plate XXXVIII, present no unusual features, except for the cast-steel

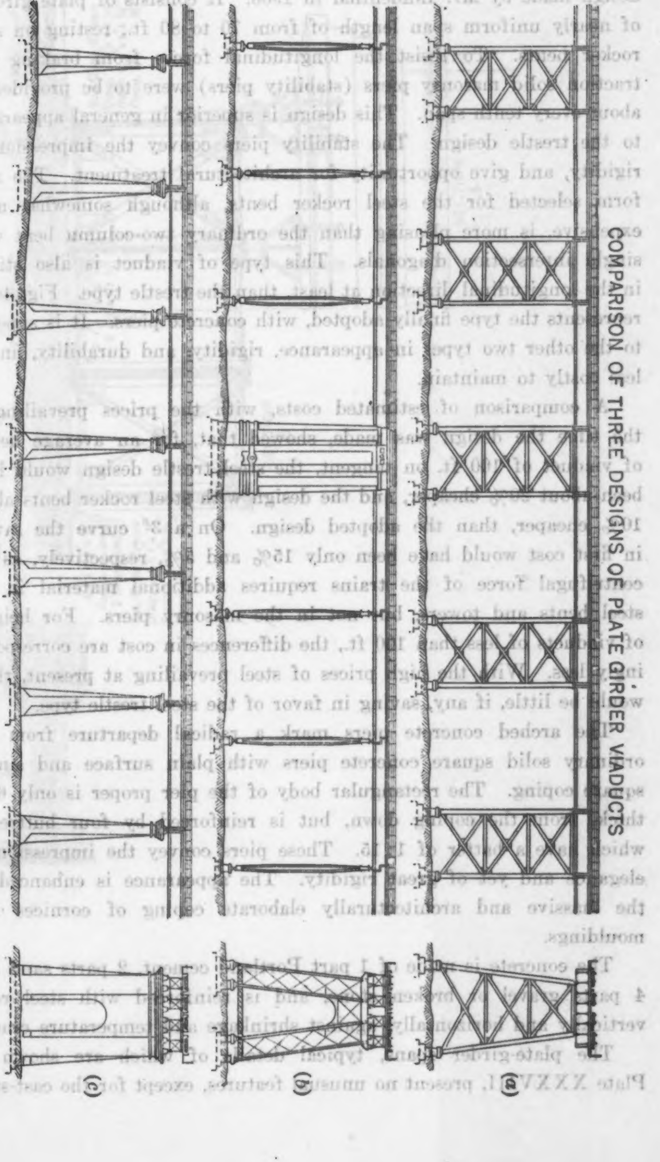

FIG. 44.
COMPARISON OF THREE DESIGNS OF PLATE GIRDER VIADUCTS

design made by Mr. Lindenthal in 1906. It consists of plate girders, of nearly uniform span length of from 70 to 80 ft., resting on steel rocker bents. To resist the longitudinal forces from braking and traction solid masonry piers (stability piers) were to be provided at about every tenth span. This design is superior in general appearance to the trestle design. The stability piers convey the impression of rigidity, and give opportunity for architectural treatment. The arch form selected for the steel rocker bents, although somewhat more expensive, is more pleasing than the ordinary two-column bent with single intersection diagonals. This type of viaduct is also stiffer, in the longitudinal direction at least, than the trestle type. Fig. 44(*c*) represents the type finally adopted, with concrete piers. It is superior to the other two types in appearance, rigidity, and durability, and is less costly to maintain.

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The plate-girder spans, typical details of which are shown in Plate XXXVIII, present no unusual features, except for the cast-steel

PLATE XXXVIII.
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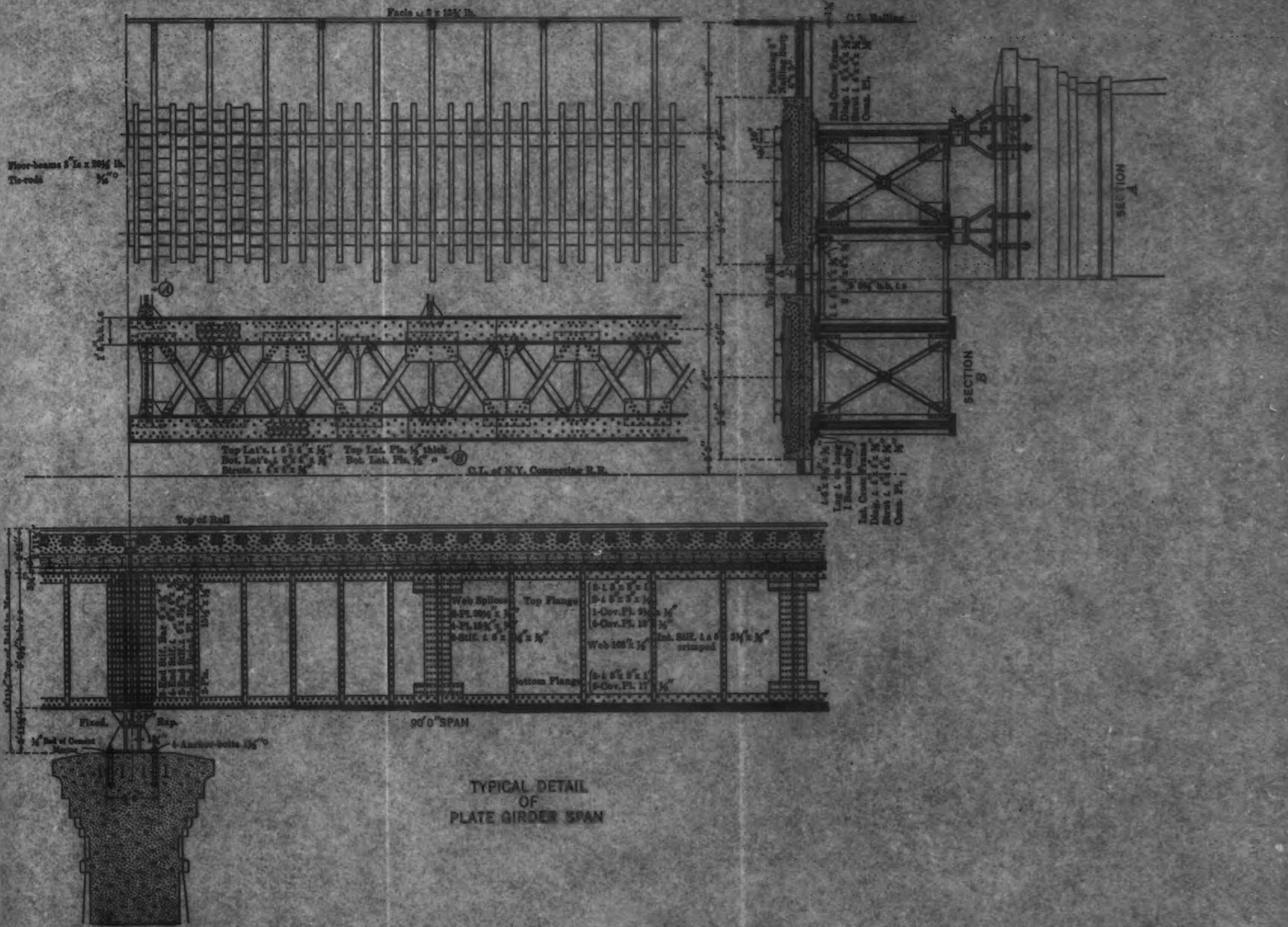
HELL GATE BRIDGE.



TYPICAL DETAIL
OF
PLATE GIRDER SPAN

bearings, which were designed to keep the reaction from one-sided loading as close as possible to the center of the pier. Special attention was also given to efficient web splices.

Erection of Plate-Girder Viaducts.

The erection of the viaducts presented no unusual difficulties, and the great number of nearly uniform spans, with the large tonnage involved, afforded opportunity for economical and rapid erection. The Bronx and Randalls Island Viaducts, for which the McClintic-Marshall Construction Company had the contract, were erected by a 50-ton steel derrick car, in some operations assisted by a 50-ton locomotive crane (Fig. 45).

All material for these viaducts was delivered over temporary tracks laid on the finished portion of the viaduct. Where possible, the girders were shipped and erected riveted up in pairs at the shop. A remarkable record was made on March 8th, 1915, when, after careful preparation, twenty-two single-track spans, with an aggregate weight of 1 504 tons, were put in place in a single 8-hour day.

A somewhat different method was used in the erection of the Wards and Long Island Viaducts, for which the American Bridge Company had the contract. After the Hell Gate Arch had been closed, and as the temporary back-stays were being dismantled, the plate girders, about 50% of which had formed part of the back-stays and counterweights, were distributed on the ground along the viaducts by using a locomotive crane running on a temporary track. The two 65-ton steel travelers, which had previously been used for the erection and dismantling of the back-stays of the arch bridge, were set up at the ends of the viaducts and proceeded toward the Hell Gate Bridge, raising the girders singly from the ground (Fig. 46).

Quantities, Weights, and Cost of Viaducts.

Table 6 gives the principal dimensions and quantities and the cost per linear foot for the different viaduct sections.

The weight of the steelwork (exclusive of I-beams in flooring), in pounds per linear foot, of single-track plate-girder spans, can be expressed approximately by the formula:

$$W = 350 - 17\, l$$

for span lengths of $l = 72$ to 94 ft.

TABLE 6.—DIMENSIONS, QUANTITIES, COST, ETC., OF VIADUCTS.

Section.	Bronx Viaduct South of 182d St.	Randalls Island Viaduct.	Wards Island Viaduct.	Long Island Viaduct.
Length of section, in feet	1 071	1 965	2 654	2 868
Approximate average height of rail above ground surface, in feet	60	80	110	90
Type of foundation	Caissons 45 to 60 feet deep.	Mostly shallow and narrow footings.	Mostly shallow and narrow footings.	Mostly deep and wide footings.
Average span length, in feet	89.0	82.0	88.5	85.5
Total quantity of concrete in piers, in cubic yards	17 500	29 800	60 300	77 900
Total weight of steelwork, in tons	4 875	7 610	11 190	11 980
Approximate cost per linear foot of viaduct, inclusive of tracks	$445	$370	$445	$500

Deck Truss Bridges of Long Island Eastern Viaduct.

Potter Avenue, and the streets between Steinway and Flushing Avenues, in Long Island City, are crossed by deck-truss bridges aggregating 1 231 ft. in length. On account of excessive span length and limited height, concrete arches, such as were built over the other streets on the Long Island Eastern Viaduct, would have been impracticable. Potter Avenue, which is 80 ft. wide, is crossed by the railroad at an angle of only 19 degrees. The distance between street lines in the direction of the tracks is about 250 ft., and the rails are only 50 ft. above street level. A skew steel bridge, about 270 ft. long, with abutments parallel to the street lines, would have been a very unsatisfactory design, and probably as expensive as the one adopted, although lighter in steelwork.

The bridge, as built, consists of three square deck-truss spans, each 135 ft. long (Plate XXXIX and Fig. 47). The two ends are supported on concrete abutments, which form a monolithic structure with the embankment side-walls. Two heavy steel rocker bents, each consisting of two columns and a cross-girder, form the intermediate supports. Two of the columns are placed in the center of the street, on permission secured from the city. Each span has four trusses, one for each track. They are $18\frac{1}{2}$ ft. deep, and 13 ft. 9 in. apart on centers. The trusses have fixed bearings on one abutment and expansion bearings on the other, the intermediate steel bents acting as rockers. The longitudinal forces from all spans, therefore, are transmitted to one end, and all temperature expansion takes place toward

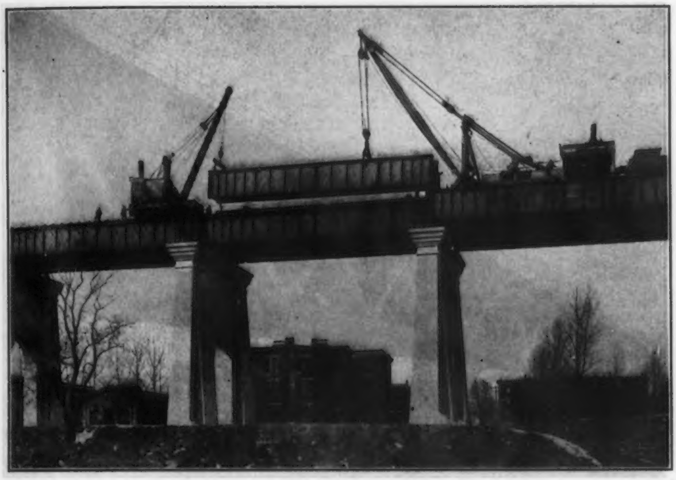


FIG. 45.—ERECTION OF RANDALLS ISLAND VIADUCT.

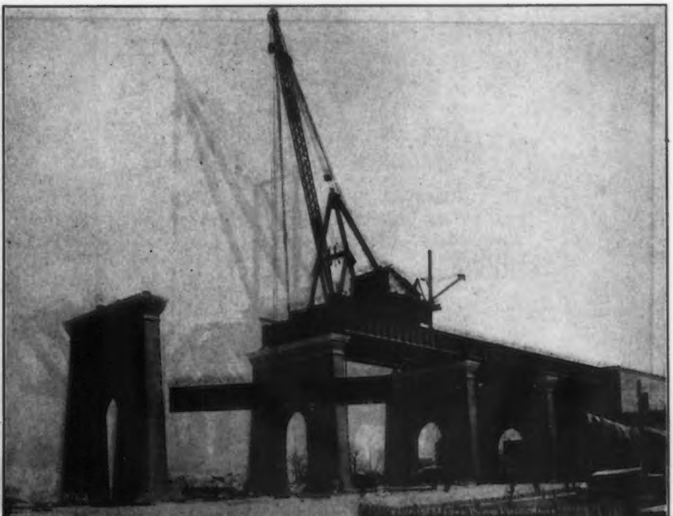


FIG. 46.—ERECTION OF LONG ISLAND VIADUCT.

FIG. 45.—ERECTION OF RANDALLS ISLAND VIADUCT.

FIG. 46.—ERECTION OF LONG ISLAND VIADUCT.

FIG. 47.—POTTER AVENUE CROSSING.

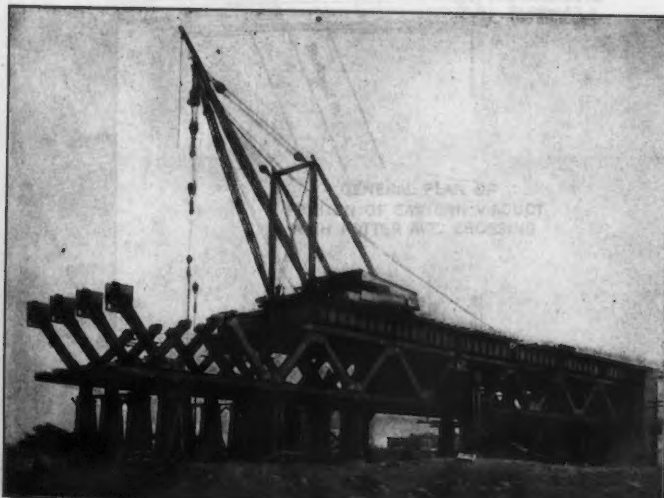


FIG. 48.—ERECTION OF POTTER AVENUE CROSSING.

FIG. 47.—POTTER AVENUE CROSSING.

FIG. 48.—ERECTION OF POTTER AVENUE CROSSING.

PLATE XXXIX.
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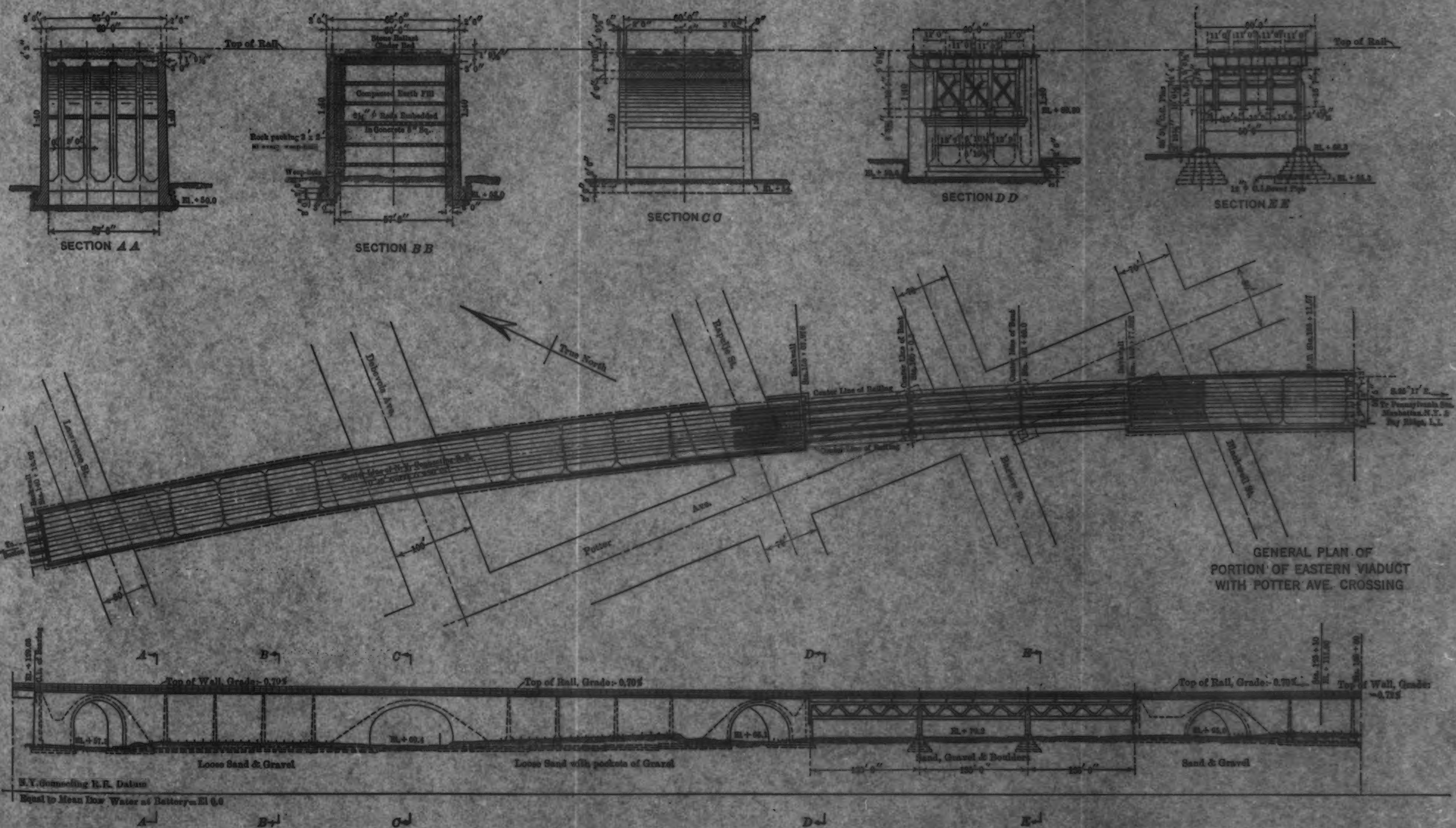

GENERAL PLAN OF
PORTION OF EASTERN VIADUCT
WITH POTTER AVE. CROSSING

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the other end. Over the steel bents the trusses are supported at their top chord points by cast-steel pin bearings, which in turn are supported by the cross-girders of the bent. Each of two adjoining trusses turns independently on its bearing, and the bottom chord member opposite the bearing is free to slide at its end, so that the trusses act as simple spans.

The columns are provided with pin bearings at the bottom, so as to be free from bending stresses from longitudinal forces or temperature expansion of the trusses. Transversely, however, the columns, with the cross-girders, form rigid portals which transmit the reactions from the lateral forces to the foundations. The cross-girders are double web-girders, 10 ft. deep, and weigh 130 tons each. They were shipped in three sections. The floor-beams rest on top of the trusses, and the stringers are framed into them. To avoid stresses in the floor-beams, due to unequal deflections of the trusses, all floor-beams except those over supports are interrupted between the two interior trusses.

The design of the truss bridges, between Steinway and Flushing Avenues, was governed by similar conditions, and is similar in every respect to that of the Potter Avenue Crossing, except that all intermediate supports are solid concrete piers instead of steel bents, and are on railroad property. The spans vary in length from 120 ft. to 165 ft. 11 in. The depth of the trusses is only 16 ft. 4 in., which was limited by the required minimum height of 16 ft. above the street level. The trusses, therefore, are unusually heavy.

All these truss bridges were erected on heavy timber bents by an 80-ton steel traveler (Fig. 48). Erection was started in October, 1914, and completed in April, 1915. Potter Avenue Crossing contains approximately 4 000 cu. yd. of concrete masonry and 3 722 tons of steel work, and the cost of construction, inclusive of tracks, was about $275 000, or $675 per lin. ft. The Steinway-Flushing Avenue Crossing contains approximately 7 600 cu. yd. of masonry and 6 526 tons of steelwork, and its cost of construction, inclusive of tracks, was about $500 000, or $565 per lin. ft.

Embankment Portion of Long Island Eastern Viaduct.

Except for the steel truss bridges over certain streets, as described before, the Eastern Viaduct, which extends from Lawrence Street to Stemler Street in Long Island City, a total length of 3 500 ft.,

consists of a novel type of embankment, from 30 to 65 ft. in height above ground. Seven streets are crossed by reinforced concrete arches which form a monolithic structure with the retaining walls of the embankment. (Plate XXXIX and Fig. 43.)

The embankment consists of two longitudinal reinforced concrete retaining walls, connected and held in relative position by horizontal steel tie-rods, which are embedded individually in a shell of concrete for protection against corrosion. These rods resist the pressure from the earth fill. For additional stability, the two walls are connected by thin cross-walls, about 50 ft. apart. The arches consist of a comparatively thin barrel reinforced by vertical ribs. The fill is mixed clay, sand, and gravel, carefully placed in 12-in. crowned layers, and thoroughly tamped, so as to form a uniform compact mass which exerts a comparatively small pressure on the retaining walls. It is thoroughly drained by chimneys of rock packing which extend along the walls from the top of the fill to the weep-holes at the bottom.

The walls and arches have perfectly plain surfaces and a simple coping. No attempt has been made at architectural treatment, because the territory in the vicinity is being built up mostly by industrial buildings which hide that portion of the railroad from prominent view. This embankment construction is considerably cheaper than the ordinary type, which consists of a fill between two independent gravity walls. For a height of 50 ft. the latter type would have cost from 30 to 40% more.

A plate-girder viaduct with concrete piers, such as was used north of Lawrence Street and over the island, would also have been more expensive than the adopted type of embankment. For heights exceeding about 65 ft., however, the plate-girder viaduct became cheaper.

The embankment contains approximately 70 000 cu. yd. of concrete masonry, 160 000 cu. yd. of earth fill, and 1 000 tons of steel reinforcement.

The cost of construction per linear foot of embankment, inclusive of tracks, was approximately \$470 for a height of 65 ft. and \$280 for a height of 35 ft., or (6.5 h — 50) dollars per square foot of elevation, h being the height of rail above the ground line.

13.—Track Floor Construction.

The franchise required a ballasted roadbed on the Bronx Section, north of Bronx Kill, and on the Long Island Section, south of Hell

Gate. A solid ballasted floor, owing to its advantages over the open tie floor, namely, more uniform roadbed, less noise and impact, smaller cost of maintenance, and greater safety in case of derailment or fire, was adopted throughout, however, except for the bascule spans of the Bronx Kill Bridge, although it involved a considerable additional initial expenditure over the open tie floor.

Various types of solid floor construction were considered. Previous to construction a wooden floor, consisting of framed, treated ties, laid closely together, appeared to be most suitable, owing to its lightness and, as it seemed then, its comparatively small cost. By the time construction started, however, the prices of framed and treated timber had increased considerably, and a more durable and fire-proof, although somewhat heavier and slightly more expensive, concrete floor construction was adopted.

A reinforced concrete slab, in which the compressive strength of the concrete and the bond between the steel rods and the concrete are relied on to resist the stresses, was not considered advisable, in view of the limited experience as regards the durability of such a construction under the heavy impact to which the floor would be subjected.

The type finally adopted (Fig. 49) consists of 8-in. I-beams, about 15 in. from center to center, placed across the two stringers or girders of each track. These beams are tied together near the bottom with ⅝-in. tie-rods, about 10 in. apart, and thus form, by themselves, a comparatively rigid steel skeleton. The axle load of 70 000 lb., plus 200% impact, was assumed as distributed over three beams, or an equivalent length of track of twice the height from top of rail to top of I-beams.

The concrete, which is placed between and from 2¾ to 3½ in. above the beams, but flush with the bottom surface of the beams, forms merely an encasement. To prevent transverse temperature cracks at the top surfaces, the concrete is further reinforced by longitudinal rods resting on, and tied to, the steel beams. The ballast is held on each side of the track by parapet walls, which are also well reinforced against temperature or shrinkage cracks. The I-beams served as ties for the construction tracks, and thereby saved the cost of temporary timber ties. Plain steel sheets, ⅛ in. thick, wedged against the bottom of the beams, constituted simple forms for the bottom surface of the concrete. The space between the concrete troughs is utilized for

footwalks and for the six-duct concrete conduit construction. Expansion joints in the concrete troughs are provided at all expansion joints in the steel superstructure. These joints are covered by T-shaped steel dams, bedded on a thin layer of asphalt.

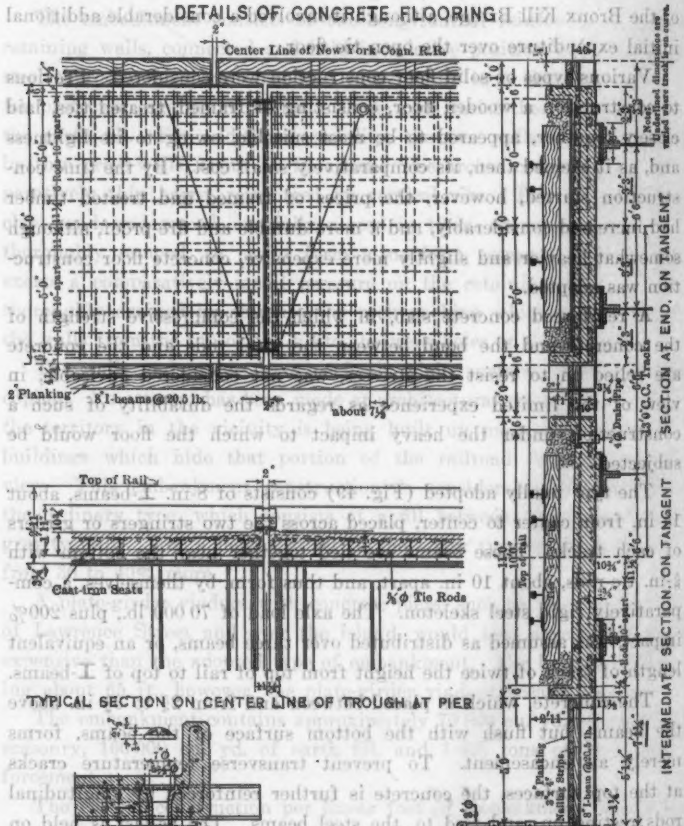

FIG. 49.

Care was taken to secure a dense concrete by using a mixture of 1 part Portland cement, 2 parts of well-graded sand, and 4 parts of broken limestone, the latter composed of 75% of ¾-in. stone and 25% of screenings. This concrete, tested on 4-in. cubes, showed an average compressive strength of about 3 500 lb. per sq. in. at the age of 28

days. The top of the concrete slab was carefully troweled to a smooth finish. No water-proofing material was placed thereon.

For efficient drainage, the top surface was given a transverse slope of 1½ in. in 10 ft., and 4-in. cast-iron drain pipes, with strainers, were placed about 15 ft. apart. On the street crossings, these pipes discharge into steel gutters which lead to 6-in. down-spouts at the piers or abutments.

The sidewalks and the walks between the tracks are of 2-in. wooden planks resting on extensions of the I-beams. The flooring contains, per linear foot of 4-track structure, 1.5 cu. yd. of concrete, 1 200 lb. of steel, and 40 ft. b. m. of timber, and cost about $53 per lin. ft. The I-beams were furnished and erected by the contractors for the steelwork. The concrete and the timber flooring were placed on contract by Fraser, Brace and Company and The Snare and Triest Company.

14.—Engineering Organization.

The New York Connecting Railroad has been built under the direction of Mr. Samuel Rea as President and Mr. A. T. County, Assistant to the President. Mr. Gustav Lindenthal, Consulting Engineer and Architect, prepared the plans for the East River Bridge Division, and, as Chief Engineer, directed their execution. During construction the Chief Engineer was assisted by an engineering staff of ninety-five members.

O. H. Ammann, M. Am. Soc. C. E., Assistant Chief Engineer, had general charge of the office, field, and inspection work, H. W. Hudson, M. Am. Soc. C. E., Construction Engineer, was in direct charge of the field operations, in which he was assisted, in the earlier stages of the work, by three Resident Engineers, George W. Philips, Assoc. M. Am. Soc. C. E., R. T. Robinson, Assoc. M. Am. Soc. C. E., and S. D. Heed, Assoc. M. Am. Soc. C. E., and later by Mr. S. D. Heed as Assistant Construction Engineer. D. B. Steinman, Assoc. M. Am. Soc. C. E., Special Assistant Engineer, attended to computations and strain measurements, and Mr. W. A. Cuenot, Assistant Engineer, to the drafting and checking of the plans and shop drawings.

A very thorough inspection was exercised over all materials. All cement which went into the work was tested at the Company's laboratory, in charge of Mr. G. B. MacWhinney, Assistant Engineer. The steel was tested and inspected at the mills and at the various shops by a corps of fifteen inspectors, directed successively by Mr. J. C.

Naegeley, as Engineer of Inspection, and Messrs. William E. Crane and R. E. McGough, as Chief Inspectors.

Conclusion.

Some of the broader engineering questions which suggest themselves in the design and execution of the structure forming the subject of this paper may be summarized as follows:

A great engineering work cannot be spontaneously created in its final, perfect form, but has to grow and develop gradually, in its entirety as well as in its constituent parts. Although the layman can only judge such a work in the light of an accomplished fact, the engineer must ever be conscious that it is only through extensive and laborious preliminary studies, and untiring efforts to improve, that he can hope to achieve a perfect work.

In the execution of a great and complex engineering or scientific undertaking, collaboration of experts in various fields is essential, but a great structure of monumental character must be the product of an individual creative and directive mind.

A great structure cannot be the result of a set of rules and specifications, nor of elaborate mathematical computations. Such a work requires wide experience and sound judgment, and therefore, should be entrusted only to engineers of high professional attainments and reputation.

Throughout this paper the importance of rigidity in bridge construction has been pointed out. Rigidity insures greater durability and safety. There are remarkable examples of structures which have stood up under excessive strains under which they would have failed had it not been for the rigidity of their members or connections. Large bridges must be built for generations to come. Engineers to-day cannot afford to build important structures cheaply, to serve their purpose for the time being, and incur the risk of having to replace them after a short period of usefulness.

Emphasis has been laid on the appearance of the structures described. Engineering structures are still regarded by many engineers as mere works of utility, which deserve no consideration in architectural or artistic treatment. So long as this opinion prevails, the Engineering Profession will not lift itself to a higher plane, and it is even running the risk of being relegated to second place—or after the architect—in the creation of such monumental structures as properly belong in its domain.

APPENDIX A

Calculation of Dead-Load, Live-Load, and Temperature Stresses in Arch Trusses of Hell Gate Bridge.

1.—Influence Line for Horizontal Reaction of 2-Hinged Arch.

In calculating the influence line for horizontal reaction, the following analytical method has been applied:

A.—Calculation of Values of Δs, $\Delta \alpha$, and Δl.— If the arch is considered a simple span fixed at A and free to move at B (Fig. 50) and if, for any one member of the truss,

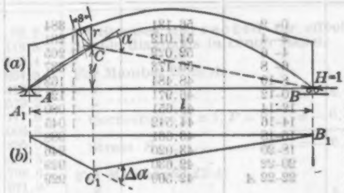

FIG. 50.

A = area of its gross section, in square inches;

s = its length, in feet;

r = its lever arm (perpendicular distance from its center of moments, C), in feet;

y = ordinate of its center of moments, above line connecting hinges, in feet;

$S_1 = \frac{y}{r}$ = stress in the member due to the sole application of a horizontal force of unity at B, and E = modulus of elasticity (30 000 000 lb. per sq. in.); then the axial deformation of the member is, in feet,

$$\Delta s = \frac{S_1 s}{A E} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

If it is assumed that only this one member is elastic, the angle, α, between the lines, $A\,C$ and $B\,C$, will change by an amount

$$\Delta \alpha = \frac{\Delta s}{r} \text{(arc measure)} \ldots\ldots\ldots\ldots\ldots (2)$$

that is, the elastic line is a triangle, $A_1\,C_1\,B_1$ (Fig. 50 (*b*)) the sides of which, $A_1\,C_1$ and $B_1\,C_1$, form the angle $\Delta \alpha$.

The point, B, moves horizontally, that is, the span length, l, changes by an amount, in feet, equal to

$$\Delta l = \frac{y}{r} \Delta s = y \Delta \alpha \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The sum, $\Sigma \Delta l$, of the values Δl, for all truss members gives the total horizontal movement of the point, B, due to the sole application of the horizontal load of unity at B.

Tables 7 and 8 show the calculation of the values, Δs, $\Delta \alpha$, and Δl, for each truss member, and also the sum, $\Sigma \Delta l$. For convenience

TABLE 7.—DETERMINATION OF VALUES

	Member.	(1) Length, s, in feet.	(2) Gross Area, A, in square inches.	(3) Lever Arm, r, in feet.	(4) Ordinate of Center of Moments, y, in feet.	(5) Stress $S_1 = \frac{y}{r}$
BOTTOM CHORD.	0- 2	56.131	1 384	−106.00	+140.00	−1.3208
	2- 4	54.012	1 281	− 88.86	+149.60	−1.6835
	4- 6	52.022	1 265	− 75.92	+162.98	−2.1461
	6- 8	50.173	1 337	− 67.76	+180.00	−2.6562
	8-10	48.484	1 163	− 62.38	+197.88	−3.1714
	10-12	46.971	1 121	− 57.39	+213.43	−3.7180
	12-14	45.651	1 080	− 52.88	+226.80	−4.2890
	14-16	44.542	1 045	− 48.92	+237.94	−4.8639
	16-18	43.661	988	− 45.60	+246.85	−5.4134
	18-20	43.020	946	− 43.01	+253.54	−5.8949
	20-22	42.630	929	− 41.21	+258.00	−6.2606
	22-22 A	42.500	929	− 40.22	+260.22	−6.5669*
TOP CHORD.	1- 3	43.571	315	+110.16	+ 86.67	+0.3328
	3- 5	44.541	315	+ 88.67	+ 70.00	+0.7814
	5- 7	45.800	315	+ 74.24	+100.00	+1.3470
	7- 9	46.089	329	+ 65.62	+126.67	+1.9303
	9-11	45.273	385	+ 59.54	+150.00	+2.5193
	11-13	44.553	385	+ 54.18	+170.00	+3.1377
	13-15	43.936	385	+ 49.60	+186.67	+3.7634
	15-17	43.424	385	+ 45.85	+200.00	+4.3651
	17-19	43.023	385	+ 43.01	+210.00	+4.8826
	19-21	42 733	385	+ 41.11	+216.67	+5.2704
	21-23	42.558	329	+ 40.16	+220.00	+5.4781
	23-23 A	42.500	329	+ 40.22	+220.00	+5.3729*

in calculation, the foregoing values have been determined in units of 1 000 E. (See Columns 6, 7, 8, and 9 of Tables 7 and 8.)

B.—Determination of Elastic Curve of Arch Truss.—As can easily be proved, the elastic line, $A_1\ C_1\ B_1$ (Fig. 50 (*b*)), assuming again only the one member elastic, is identical with the moment diagram of a simple span, $A\ B$, due to the sole application of a vertical load, $\Delta\ \alpha$, at the center of moments, C, of the member in question.

From this rule, if applied to every truss member, it follows that the elastic line of the arch truss due to the sole application of a horizontal load of unity at B is identical with the moment diagram due to the application of the values, $\Delta\ \alpha$, as vertical loads, called "Elastic Loads", at the respective centers of moments of the truss members.

The center of moments of a chord member, and therefore the corresponding elastic load, $\Delta\ \alpha$, is always at a panel point.

The center of moments of a web member, in general, is not at a panel point, nor is it always within the span length, and, therefore, it

Δs, $\Delta \alpha$, and Δl, for Chord Members.

(6) $1\,000\,E\,\Delta s = 1\,000\,\frac{S_1\,s}{A}$	(7) $1\,000\,E\,\Delta \alpha = 1\,000\,\frac{E\,\Delta s}{r}$	(8) $1\,000\,E\,\Delta l = \frac{y}{r}\,1\,000\,E\,\Delta s = y\,1\,000\,E\,\Delta \alpha$	(9) Notes.
− 53.57	+ 0.505	+ 70.7	* Stresses corrected for effect of double diagonals in center panel. For Member 22-22 *A*: $\frac{y}{r}$ .... $= -6.4699$ Correction ($H = 1, P = 0$)... $= -0.0970$ Stress S_1 .... $= -6.5669$ For Member 23-23 *A*: $\frac{y}{r}$ .... $= +5.4699$ Correction ($H = 1, P = 0$)... $= -0.0970$ Stress S_1 .... $= +5.3729$
− 70.98	+ 0.799	+ 119.5	
− 88.26	+ 1.163	+ 189.4	
− 99.68	+ 1.471	+ 264.8	
− 132.21	+ 2.119	+ 419.3	
− 156.19	+ 2.715	+ 579.5	
− 181.29	+ 3.428	+ 777.6	
− 206.90	+ 4.229	+ 1 006.3	
− 239.22	+ 5.246	+ 1 295.0	
− 268.08	+ 6.283	+ 1 580.3	
− 287.29	+ 6.971	+ 1 798.6	
− 300.42	+ 2 × 3.735	+ 2 × 971.9	
	$1\,000\,E\,\frac{\Sigma\,\Delta l}{2} =$	+ 9 071.9	
$1\,000\,E\,\Sigma\,\Delta l$ for bottom chords =		+ 18 143.8	
+ 46.04	+ 0.418	+ 15.3	
+ 111.63	+ 1.259	+ 88.1	
+ 195.85	+ 2.638	+ 263.8	
+ 270.41	+ 4.121	+ 522.0	
+ 296.25	+ 4.976	+ 746.4	
+ 363.10	+ 6.702	+ 1 139.3	
+ 429.48	+ 8.659	+ 1 616.3	
+ 491.99	+ 10.731	+ 2 146.1	
+ 545.62	+ 12.686	+ 2 664.0	
+ 584.99	+ 14.230	+ 3 083.2	
+ 708.62	+ 17.645	+ 3 881.9	
+ 693.95	+ 2 × 8.627	+ 2 × 1 897.5	
	$1\,000\,E\,\frac{\Sigma\,\Delta l}{2} =$	+ 18 068.9	
$1\,000\,E\,\Sigma\,\Delta l$ for top chords =		+ 36 127.8	

is more convenient to substitute for the load, $\Delta \alpha$, applied at the center of moments of a web member, two vertical panel loads as follows:

Take, for instance, the diagonal, $D\,E$ (Fig. 51), the center of moments of which is at C. The elastic line, considering only $D\,E$ elastic, is a broken line, $A_1\,D_1\,E_1\,B_1$, the segments of which, $A_1\,D_1$ and $B_1\,E_1$, intersect at C_1 vertically below C, and enclose the angle, $\Delta \alpha$. It can easily be proved that this line is identical with the moment diagram due to the vertical loads, $\Delta \alpha' = \Delta \alpha \left[\frac{x}{\lambda} - (n + 1)\right]$ and $\Delta \alpha'' = \Delta \alpha \left(\frac{x}{\lambda} - n\right)$, applied at the panel points, D and E,

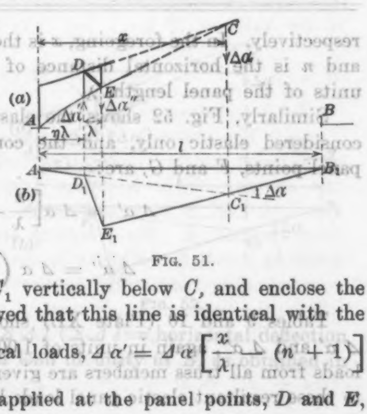

FIG. 51.

TABLE 8.—DETERMINATION OF VALUES,

	Member.	(1) Length, s, in feet.	(2) Gross area, A, in square inches.	(3) Lever arm, r, in feet.	(4) Ordinate of center of moments, y, in feet.	(5) Stress, $S_1 = \frac{y}{r}$
DIAGONALS.	1- 2	111.732	235.5	− 222.2	+ 159.6	− 0.8535
	3- 4	90.235	228.4	− 247.2	+ 224.9	− 0.9097
	5- 6	75.937	201.8	− 321.8	+ 285.6	− 0.8875
	7- 8	68.197	129.8	− 401.5	+ 341.4	− 0.8504
	9-10	68.984	129.8	− 387.7	+ 341.4	− 0.8805
	11-12	60.765	129.8	− 380.1	+ 341.3	− 0.8981
	13-14	58.454	129.8	− 383.1	+ 341.3	− 0.8907
	15-16	56.971	168.8	− 405.2	+ 341.2	− 0.8422
	17-18	56.251	168.8	− 465.6	+ 341.5	− 0.7335
	19-20	56.266	196.0	− 615.9	+ 341.5	− 0.5545
	21-22	57.011	196.0	− 1 113.6	+ 340.5	− 0.3058
	23-22	58.514	129.8			+ 0.1335*
VERTICALS.	0- 1	140.000	312.0 } Average	+ 42.5	+ 36.7	+ 0.8628
	2- 3	112.933	255.0 } Average	+ 202.2	+ 195.3	+ 0.9656
	4- 5	92.930	235.0 } Average	+ 237.3	+ 237.3	+ 1.0013
	6- 7	80.000	151.0	+ 354.3	+ 322.3	+ 0.9097
	8- 9	71.163	137.0	+ 549.6	+ 428.4	+ 0.7795
	10-11	63.430	126.1	+ 612.7	+ 438.3	+ 0.7154
	12-13	56.800	126.1	+ 732.2	+ 457.1	+ 0.6244
	14-15	51.273	126.1	+ 993.7	+ 498.4	+ 0.5016
	16-17	46.850	126.1	+ 1 826.7	+ 629.8	+ 0.3448
	18-19	43.540	126.1	− 80 442.0	+ 12 410.	+ 0.1543
	20-21	41.333	126.1	− 1 556.5	+ 94.6	− 0.0608
	22-23	40.220	126.1	− 769.7	+ 220.0	− 0.3776†

respectively. In the foregoing, x is the horizontal distance of C from A, and n is the horizontal distance of the panel point, D, from A, in units of the panel length, λ.

Similarly, Fig. 52 shows the elastic line, if the vertical, $F\,F'$, is considered elastic only, and the corresponding elastic loads at the panel points, F and G, are:

$$\varDelta\,\alpha' = \varDelta\,\alpha\left[\frac{x}{\lambda} - (n+1)\right]$$

$$\varDelta\,\alpha'' = \varDelta\,\alpha\left(\frac{x}{\lambda} - n\right).$$

Tables 9 and 10 (Plate XL) show the calculation of the values, $\varDelta\,\alpha'$ and $\varDelta\,\alpha''$, again in units of 1 000 E. The resultant elastic panel loads from all truss members are given in Table 12. The moments due to these resultant elastic panel loads have been determined analytically

Δs, $\Delta \alpha$, AND Δl, FOR WEB MEMBERS.

(6)	(7)	(8)	(9)
$1000\,E\,\Delta s = 1\,000\,\dfrac{S_1 s}{A}$	$1\,000\,E\,\Delta\alpha = 1\,000\,\dfrac{E\,\Delta s}{r}$	$1\,000\,E\,\Delta l = \dfrac{y}{r}\,1\,000\,E\,\Delta s = y\,1\,000\,E\,\Delta\alpha$	Notes.
− 404.9	+ 1.8224	+ 345.6	* The stress, S_1, in the center diagonals has no influence on the deflection line.
− 359.4	+ 1.4551	+ 327.0	† The stress in 22-23 is corrected for effect of double diagonals in center panel.
− 334.0	+ 1.0879	+ 296.4	
− 446.8	+ 1.1126	+ 379.9	
− 434.0	+ 1.1194	+ 382.2	
− 420.4	+ 1.1062	+ 377.6	$\dfrac{y}{r}$ .... = −0.2858
− 401.1	+ 1.0470	+ 357.3	Correction ($H = 1$, $P = 0$)... = −0.0918
− 284.3	+ 0.7016	+ 239.4	
− 244.4	+ 0.5250	+ 179.3	Stress S_1.................. = −0.3776
− 159.2	+ 0.2585	+ 88.3	
− 88.7	+ 0.0797	+ 27.1	
	$1\,000\,E\,\dfrac{\Sigma\,\Delta l}{2}$ =	+ 3 000.1	
$1\,000\,E\,\Sigma\,\Delta l$ for diagonals =		+ 6 000.2	
+ 387.1	+ 9.109	+ 334.0	
+ 427.6	+ 2.114	+ 412.9	
+ 396.0	+ 1.671	+ 396.5	
+ 482.0	+ 1.360	+ 438.5	
+ 404.9	+ 0.800	+ 315.6	Summary of values....... $1\,000\,E\,\Sigma\,\Delta l$
+ 359.9	+ 0.737	+ 257.5	
+ 281.2	+ 0.384	+ 175.6	Bottom chord .... = 18 143.8
+ 203.9	+ 0.205	+ 102.3	Top chord.................. = 36 127.8
+ 128.1	+ 0.0701	+ 44.2	Diagonals.................. = 6 000.2
+ 53.3	− 0.00066	+ 8.2	Verticals.................. = 5 041.8
− 19.9	+ 0.0128	+ 1.2	
− 120.4	+ 0.156	+ 34.4	Total..........$1\,000\,E\,\Sigma\,\Delta l$ = 65 313.6
	$1\,000\,E\,\dfrac{\Sigma\,\Delta l}{2}$ =	+ 2 520.9	
$1\,000\,E\,\Sigma\,\Delta l$ for verticals =		+ 5 041.8	

in Table 12 by the usual method of shears and moment increments. The ordinate, δ, of the elastic line at any panel point, is equal to the corresponding moment from the elastic panel loads.

C.—Determination of Influence Line for Horizontal Reaction.—The influence ordinates for the horizontal reaction were determined in Table 12 by dividing the corresponding ordinates, δ, of the elastic line by the constant, $\Sigma\,\Delta l$ ($\Sigma\,\Delta l$ = horizontal deflection of the point, B, due to a horizontal load of unity at B, as obtained in Table 8, Column 9).

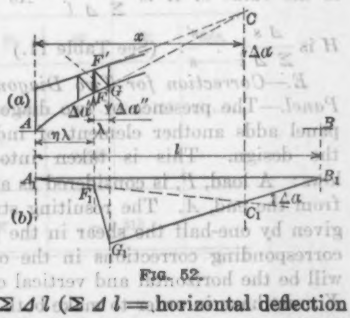

FIG. 52.

This is derived as follows: Assume the arch to be a simple span, $A\ B$, fixed at A and free to move at B (Fig. 53). According to Maxwell's Principle of Reciprocity, the vertical deflection, δ, at any point, C, due to a horizontal force of unity at B (Fig. 53 (*a*) and (*b*)), is equal to the horizontal deflection, ϵ, of the point, B, due to a vertical force of unity at C (Fig. 53 (*c*)), that is $\delta = \epsilon$.

In order to transform the simple span into a two-hinged arch, the horizontal reaction, H, has to overcome the horizontal deflection, ϵ, and we have, therefore, the relation, $\dfrac{H}{H \text{ unity}} = \dfrac{\epsilon}{\Sigma \varDelta\, l}$; or, as $\epsilon = \delta$, $H = \dfrac{\delta}{\Sigma \varDelta\, l}$.

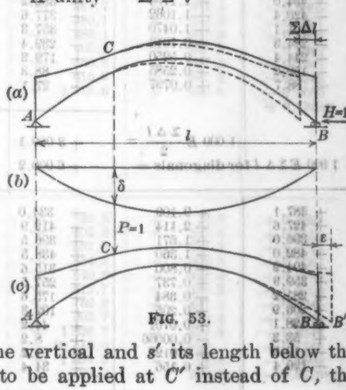

FIG. 53.

D.—Corrections for the Verticals, 0-1, 2-3, 4-5.—In the foregoing it has been assumed that the vertical load of unity is applied at the bottom chord panel points. As the live load is actually applied at the floor level, the following correction has to be made in the influence ordinates for H below the panel points, 0, 2, and 4.

Let s be the total length of the vertical and s' its length below the floor (Fig. 54). As the load is to be applied at C' instead of C, the numerator, δ, in the foregoing formula, $H = \dfrac{\delta}{\Sigma \varDelta\, l}$, must be corrected to represent the deflection of C' instead of C. If δ is the deflection at C, $\delta + \varDelta\, s'$ is the deflection at C'. The resulting change in the value of H is $\dfrac{\varDelta\, s'}{\Sigma \varDelta\, l}$. As $\varDelta\, s' = \varDelta\, s \cdot \dfrac{s'}{s}$, the correction for H is $\dfrac{\varDelta\, s}{\Sigma \varDelta\, l} \cdot \dfrac{s'}{s}$. (See Table 11.)

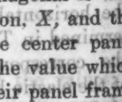

FIG. 54.

E.—Correction for Two Diagonals in the Center Panel.—The presence of two diagonals in the center panel adds another element of indeterminateness to the design. This is taken into account as follows: A load, P, is considered as acting at any point distant m panels from the end, A. The resulting stress in each center diagonal is that given by one-half the shear in the panel plus a correction, X, and the corresponding corrections in the other members of the center panel will be the horizontal and vertical components of X. The value which X must have in order to make both diagonals fit into their panel frame is then given by writing out and solving the equation, $\varDelta d_1 + \varDelta d_2 = (\varDelta u + \varDelta l)$ as $B + (\varDelta v_1 + \varDelta v_2)$ sin. B, where $\varDelta u$, $\varDelta l$, $\varDelta v$, and $\varDelta d$

TABLE 9.—DETERMINATION OF ELASTIC PANEL LOADS, $\Delta\alpha' + \Delta\alpha''$, FOR DIAGONALS.

Panel length, $\lambda = 42.5$ ft.

Member.	(1) 1-2		(2) 3-4		(3) 5-6		(4) 7-8		(5) 9-10		(6) 11-12		(7) 13-14		(8) 15-16		(9) 17-18		(10) 19-20		(11) 21-22		(12) 23-23A	
1 000 $E \Delta a$ from Table 8	+ 1.8224		+ 1.4551		+ 1.0879		+ 1.1126		+ 1.1194		+ 1.062		+ 1.0470		+ 0.7016		+ 0.5250		+ 0.2585		+ 0.0707			
x	+ 219.88		+ 262.45		+ 390.45		+ 512.24		+ 561.12		+ 619.10		+ 691.75		+ 790.20		+ 941.55		+ 1 230.80		+ 2 008.08		∞	
n	0		1		2		3		4		5		6		7		8		9		10		11	
Panel point.	0	2	2	4	4	6	6	8	8	10	10	12	12	14	14	16	16	18	18	20	20	22		
1 000 $E \Delta a'$ $= 1\,000\, E \Delta a \left[\frac{x}{\lambda} - (n+1)\right]$	− 7.608	....	− 6.755	....	− 6.430	....	− 8.962	....	− 9.182	....	− 9.477	....	− 9.712	....	− 7.432	....	− 6.906	....	− 4.840	....	− 2.870			
1 000 $E \Delta a''$ $= 1\,000\, E \Delta a \left[\frac{x}{\lambda} - n\right]$	....	+ 9.425	....	+ 8.216	....	+ 7.460	....	+ 10.076	....	+ 10.302	....	+ 10.583	....	+ 10.759	....	+ 8.188	....	+ 7.431	....	+ 5.090	....	+ 2.959		
Total elastic panel load, 1 000 $E [\Delta a' + \Delta a'']$	− 7.608	+ 2.670		+ 1.786		− 1.502		+ 0.894		+ 0.825		+ 0.871		+ 3.327		+ 1.287		+ 2.591		+ 2.220		+ 2.959		

TABLE 10.—DETERMINATION OF ELASTIC PANEL LOADS, $\Delta\alpha' + \Delta\alpha''$, FOR VERTICALS.

Panel length, $\lambda = 42.5$ ft.

Member.	(1) 0-1		(2) 2-3		(3) 4-5		(4) 6-7		(5) 8-9		(6) 10-11		(7) 12-13		(8) 14-15		(9) 16-17		(10) 18-19		(11) 20-21		(12) 22-23	
1 000 $E \Delta a$ from Table 8	+ 9.1090		+ 2.1145		+ 1.6708		+ 1.3605		+ 0.7367		+ 0.5874		+ 0.3341		+ 0.2055		+ 0.0701		− 0.0007		+ 0.0126		+ 0.1585	
x	+ 42.50		+ 244.74		+ 321.98		+ 461.77		+ 719.61		+ 825.18		+ 967.16		+ 1 291.22		+ 2 116.72		− 80 066.0		− 1 131.54		− 308.81	
n	0		1		2		3		4		5		6		7		8		9		10		11	
Panel point.	0	2	2	4	4	6	6	8	8	10	10	12	12	14	14	16	16	18	18	20	20	22	22	22A
1 000 $E \Delta a'$ $= 1\,000\, E \Delta a \left[\frac{x}{\lambda} - (n+1)\right]$	0	....	− 7.947	....	− 7.646	....	− 9.980	....	− 8.790	....	− 7.880	....	− 6.233	....	− 4.593	....	− 2.944	....	− 1.254	....	+ 0.482	....	+ 0.156	....
1 000 $E \Delta a''$ $= 1\,000\, E \Delta a \left[\frac{x}{\lambda} - n\right]$	....	+ 9.109	....	+ 10.062	....	+ 9.315	....	+ 11.341	....	+ 9.527	....	+ 8.467	....	+ 6.617	....	+ 4.798	....	+ 3.014	....	+ 1.255	....	− 0.469	....	....
Total elastic panel load, 1 000 $E [\Delta a' + \Delta a'']$	0	+ 1.162		+ 2.416		− 0.665		+ 2.551		+ 1.647		+ 2.234		+ 2.024		+ 1.854		+ 1.760		+ 1.737		− 0.313		

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TABLE 11.—CORRECTION OF INFLUENCE ORDINATES FOR HORIZONTAL REACTION DUE TO LOADS APPLIED TO VERTICALS AT FLOOR LEVEL.

$1\,000\ E\,\Sigma\,\Delta\,l = 65\,313.6.$

Member.	(1) Total length, s, in feet.	(2) Length below floor, s' in feet.	(3) $1\,000\,E\,\Delta\,s = \frac{1\,000\,S\,s}{A}$. From Table 8.	(4) Ratio $\frac{A}{A'}$.	(5) Correction. $\frac{1\,000\,E\,\Delta\,s}{1\,000\,E\,\Sigma\,\Delta\,l} \times \frac{s'}{s} \times \frac{A}{A'}$.
0-1	140.00	95	387.13	$\frac{312}{315} = 0.99$	0.00396
2-3	112.98	60	427.64	$\frac{255}{281} = 0.91$	0.00315
4-5	92.95	26	395.95	$\frac{235}{267} = 0.88$	0.00149

denote the elastic elongation of the top, bottom, vertical, and diagonal members, respectively, of the center panel, and is the inclination of the diagonals to the horizontal. We thus obtain

$$X = 0.1335\ H - 0.01538\ m\ P.$$

Consequently, for the sole application of $H = 1$, $X = +\ 0.1335$ is the stress in each center diagonal, and the resulting corrections in the other members of the center panel are included in Tables 7 and 8. The corrected value of the influence line for H is thus found, although the effect on H of the foregoing correction proves to be quite negligible.

Substituting the resulting values of H with the corresponding values of m in the foregoing equation, we obtain the influence values of X for a unit load at the successive panel points of the span. These values are tabulated as the influence ordinates for the center diagonal in Table 15 (Plate XLII), and the corresponding corrections are tabulated for the other members of the center panel in Tables 13 and 14 (Plate XLI), and Table 16 (Plate XLII). All the remaining members of the truss are unaffected by the double center diagonals.

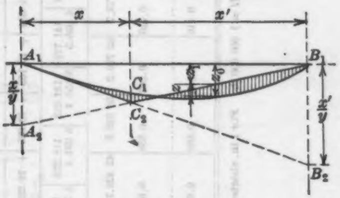

FIG. 55.

2.—Influence Lines for Arch Truss Members.

The influence ordinates for the stresses in the members of the arch truss were determined analytically as follows:

Assuming the truss to be a simple span, the influence line for the stress in any member has two straight segments, $A_1\ B_2$ and $B_1\ A_2$ (Fig. 55), which intersect at C_2 vertically below the center of moments, C, of the member, and have the ordinates, $A_1, A_2 = \frac{x}{r}$, and $B_1, B_2 = \frac{x'}{r}$.

TABLE 12.—DETERMINATION OF INFLUENCE ORDINATES FOR HORIZONTAL REACTION.

Panel Length, λ = 42.5 ft. $1\ 000\ E\ \Sigma\ \Delta\ l$ = 65 313.6.

			(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
		Panel point.	0	2	4	6	8	10	12	14	16	18	20	22	Sum of elastic panel loads for ½ truss.
Elastic panel loads.	$1\ 000\ E\ \Delta\ a$.	Bottom chord. From Table 7	....	+ 0.799	+ 1.163	+ 1.471	+ 2.19	+ 2.715	+ 3.428	+ 4.229	+ 5.246	+ 6.233	+ 6.971	+ 3.735	
		Top chord. From Table 7.	....	+ 0.418	+ 1.259	+ 2.638	+ 4.121	+ 4.976	+ 6.702	+ 8.659	+ 10.731	+ 12.686	+ 14.230	{ + 17.645 / + 8.627	
	$1\ 000\ E\ (\Delta\ a' + \Delta\ a'')$	Diagonals. From Table 9.	....	+ 2.670	+ 1.788	− 1.502	+ 0.898	+ 0.825	+ 0.871	+ 3.327	+ 1.227	+ 2.591	+ 2.230	+ 2.950	
		Verticals. From Table 9.	....	+ 1.162	+ 2.416	− 0.665	+ 2.551	+ 1.647	+ 2.234	+ 2.024	+ 1.854	+ 1.760	+ 1.737	− 0.313	
		Total elastic panel load...	....	+ 5.049	+ 6.626	+ 0.942	+ 9.684	+ 10.163	+ 13.235	+ 18.239	+ 19.058	+ 23.270	+ 25.158	+ 32.658	+ 165.077
Shear = V. Moment increment = $V\lambda$			165.077 7 015.9	160.028 6 801.3	153.402 6 519.5	151.460 6 437.0	141.776 6 025.5	131.618 5 593.4	118.378 5 031.1	100.139 4 255.9	81.081 3 445.9	57.811 2 457.0	32.653 1 387.8		
Moment = ordinate of elastic line = $1\ 000\ E\ \delta$			0	7 015.9	13 817.2	20 336.7	26 773.7	32 799.2	38 392.6	43 423.7	47 679.6	51 125.5	53 582.5	54 970.3	
Influence ordinates for horizontal reaction = $\frac{1\ 000\ E\ \delta}{1\ 000\ E\ \Sigma\ \Delta\ l}$			0	0.1074	0.2115	0.311	0.410	0.502	0.588	0.665	0.730	0.783	0.820	0.841	Sum of ordinates for ½ truss.
Correction for loads at floor level. From Table 11.			0.0040	0.0039	0.0015	....	....	....	....	....	....	....	....	....	
Total influence ordinates for horizontal reaction.			0.004	0.111	0.213	0.311	0.410	0.502	0.588	0.665	0.730	0.783	0.820	0.841	5.978

NOTE.—$1\ 000\ E\ \Sigma\ \Delta\ l$ = 65 313.6, therefore, $\frac{65\ 313.6 \times 12}{30\ 000}$ = 26.125 in., shortening for H = 1 000 000 lb. For 1 in. shortening, $H = \frac{1\ 000\ 000}{26.125}$ = 38 278 lb.

TABLE 13.—Determination of Influence Ordinates.—For Bottom Chord. Live Load per Panel = 510 000 lb. = 1 000

Member.	(1) X, in feet.	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0	2	4	6	8	10	12	14	16	18	20	22	22	20	18	16
							Panel Points.—All Values in These Columns Are Influence Ordinates, and Are Expressed in Thou															
Influence ordinate. Hor. reaction......	....	....	....	....	....	Z_0	+ 4	+ 111	+ 213	+ 311	+ 410	+ 502	+ 588	+ 665	+ 730	+ 783	+ 820	+ 841	+ 841	+ 820	+ 783	+ 730
0-2............	0	977.5	0.000	— 6.962	— 1.321	Z_1	0															
						$Z = Z_0 + Z_1$	+ 4	+ 111	+ 213	+ 311	+ 410	+ 502	+ 588	+ 665	+ 730	+ 783	+ 820	+ 841	+ 841	+ 820	+ 783	+ 730
2-4............	42.5	935.0	— 0.284	— 6.250	— 1.694	Z_1	0	— 272	— 259	— 247	— 235	— 222	— 210	— 198	— 185	— 173	— 161	— 148	— 136	— 124	— 111	— 99
						$Z = Z_0 + Z_1$	+ 4	— 161	— 46	+ 64	+ 175	+ 280	+ 378	+ 467	+ 545	+ 610	+ 659	+ 693	+ 705	+ 696	+ 672	+ 631
4-6............	85.0	892.5	— 0.533	— 5.478	— 2.146	Z_1	0	— 238	— 476	— 454	— 431	— 408	— 386	— 363	— 340	— 318	— 295	— 272	— 250	— 227	— 204	— 182
						$Z = Z_0 + Z_1$	+ 4	— 127	— 263	— 143	— 21	+ 94	+ 202	+ 302	+ 390	+ 465	+ 525	+ 569	+ 591	+ 593	+ 579	+ 548
6-8............	127.5	850.0	— 0.708	— 4.722	— 2.656	Z_1	0	— 205	— 411	— 616	— 585	— 554	— 524	— 493	— 462	— 431	— 400	— 370	— 339	— 308	— 277	— 246
						$Z = Z_0 + Z_1$	+ 4	— 91	— 198	— 305	— 175	— 52	+ 64	+ 172	+ 268	+ 352	+ 420	+ 471	+ 502	+ 512	+ 506	+ 484
8-10............	170.0	807.5	— 0.859	— 4.082	— 3.171	Z_1	0	— 177	— 355	— 532	— 710	— 672	— 635	— 598	— 560	— 523	— 486	— 448	— 411	— 374	— 336	— 299
						$Z = Z_0 + Z_1$	+ 4	— 66	— 142	— 221	— 300	— 170	— 47	+ 67	+ 170	+ 260	+ 334	+ 393	+ 430	+ 446	+ 447	+ 431
10-12............	212.5	765.0	— 0.996	— 3.584	— 3.719	Z_1	0	— 156	— 312	— 467	— 623	— 779	— 736	— 693	— 649	— 606	— 563	— 519	— 476	— 433	— 390	— 346
						$Z = Z_0 + Z_1$	+ 4	— 45	— 98	— 156	— 213	— 277	— 148	— 28	+ 81	+ 177	+ 257	+ 322	+ 365	+ 387	+ 393	+ 384
12-14............	255.0	722.5	— 1.124	— 3.186	— 4.289	Z_1	0	— 139	— 277	— 416	— 554	— 692	— 831	— 782	— 733	— 684	— 635	— 587	— 538	— 489	— 440	— 391
						$Z = Z_0 + Z_1$	+ 4	— 28	— 64	— 105	— 144	— 190	— 243	— 117	— 3	+ 99	+ 185	+ 254	+ 303	+ 331	+ 343	+ 339
14-16............	297.5	680.0	— 1.250	— 2.858	— 4.864	Z_1	0	— 124	— 249	— 373	— 497	— 621	— 746	— 870	— 815	— 761	— 707	— 652	— 598	— 544	— 489	— 435
						$Z = Z_0 + Z_1$	+ 4	— 13	— 36	— 62	— 87	— 119	— 158	— 205	— 85	+ 22	+ 113	+ 189	+ 243	+ 276	+ 294	+ 295
16-18............	340.0	637.5	— 1.377	— 2.582	— 5.418	Z_1	0	— 112	— 225	— 337	— 449	— 561	— 674	— 786	— 898	— 838	— 779	— 719	— 659	— 599	— 539	— 479
						$Z = Z_0 + Z_1$	+ 4	— 1	— 12	— 26	— 39	— 59	— 86	— 121	— 168	— 55	+ 41	+ 122	+ 182	+ 221	+ 244	+ 251
18-20............	382.5	595.0	— 1.509	— 2.347	— 5.895	Z_1	0	— 102	— 204	— 306	— 408	— 510	— 612	— 714	— 816	— 918	— 853	— 787	— 722	— 656	— 590	— 525
						$Z = Z_0 + Z_1$	+ 4	+ 9	+ 9	+ 5	+ 2	— 8	— 24	— 49	— 86	— 135	— 33	+ 54	+ 119	+ 164	+ 193	+ 205
20-22............	425.0	552.5	— 1.647	— 2.142	— 6.261	Z_1	0	— 93	— 186	— 279	— 372	— 466	— 559	— 652	— 745	— 838	— 931	— 859	— 788	— 716	— 645	— 573
						$Z = Z_0 + Z_1$	+ 4	+ 18	+ 27	+ 32	+ 38	+ 36	+ 29	+ 13	— 15	— 55	— 111	— 13	+ 53	+ 104	+ 138	+ 157
						Z_1	0	— 82	— 163	— 245	— 327	— 408	— 490	— 572	— 653	— 735	— 817	— 898	— 898	— 817	— 735	— 653
22-22A..........	488.75	488.75	— 1.878	— 1.878	— 6.470	Correct Z_2	+0.06	— 0.07	— 0.37	— 0.52	— 0.75	— 1.10	— 1.55	— 2.18	— 2.86	— 3.81	— 4.97	— 6.37	— 6.37	— 4.97	— 3.81	— 2.86
						$Z = Z_0 + Z_1 + Z_2$	+ 5	+ 29	+ 50	+ 65	+ 82	+ 93	+ 96	+ 91	+ 74	+ 44	— 2	— 63	— 63	— 2	+ 44	+ 74

TABLE 14.—Determination of Influence Ordinates.—For Top Chord. Live Load per Panel = 510 000 lb. = 1 000 P

Member.	(1) X, in feet.	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0	2	4	6	8	10	12	14	16	18	20	22	22	20	18	16
							Panel Points.—All Values in These Columns Are Influence Ordinates, and Are Expressed in Tho															
Influence ordinate. Hor. reaction......	....	....	....	....	....	Z_0	+ 4	+ 111	+ 213	+ 311	+ 410	+ 502	+ 588	+ 665	+ 730	+ 783	+ 820	+ 841	+ 841	+ 820	+ 783	+ 730
1-3............	42.5	935.0	— 1.159	— 25.500	+ 0.833	Z_1	0	— 1 109	— 1 059	— 1 008	— 958	— 907	— 857	— 807	— 756	— 706	— 655	— 605	— 555	— 504	— 454	— 403
						$Z = Z_0 + Z_1$	+ 4	— 998	— 846	— 697	— 548	— 405	— 269	— 142	— 26	+ 77	+ 165	+ 236	+ 286	+ 316	+ 329	+ 327
3-5............	85.0	892.5	— 1.214	— 12.750	+ 0.789	Z_1	0	— 554	— 1 109	— 1 056	— 1 008	— 950	— 898	— 845	— 792	— 739	— 686	— 634	— 581	— 528	— 475	— 422
						$Z = Z_0 + Z_1$	+ 4	— 443	— 896	— 745	— 598	— 448	— 310	— 180	— 62	+ 44	+ 134	+ 207	+ 260	+ 292	+ 308	+ 308
5-7............	127.5	850.0	— 1.275	— 8.500	+ 1.347	Z_1	0	— 370	— 739	— 1 109	— 1 058	— 998	— 942	— 887	— 832	— 776	— 721	— 665	— 610	— 554	— 499	— 444
						$Z = Z_0 + Z_1$	+ 4	— 259	— 526	— 798	— 648	— 496	— 354	— 222	— 102	+ 7	+ 99	+ 176	+ 231	+ 266	+ 284	+ 286
7-9............	170.0	807.5	— 1.342	— 6.375	+ 1.930	Z_1	0	— 277	— 554	— 832	— 1 109	— 1 050	— 992	— 934	— 876	— 817	— 759	— 700	— 642	— 584	— 525	— 467
						$Z = Z_0 + Z_1$	+ 4	— 166	— 341	— 521	— 699	— 548	— 404	— 269	— 146	— 34	+ 61	+ 141	+ 199	+ 236	+ 258	+ 263
9-11............	212.5	765.0	— 1.417	— 5.100	+ 2.519	Z_1	0	— 222	— 443	— 665	— 887	— 1 109	— 1 047	— 986	— 924	— 862	— 801	— 739	— 678	— 616	— 554	— 493
						$Z = Z_0 + Z_1$	+ 4	— 111	— 230	— 354	— 477	— 607	— 459	— 321	— 194	— 79	+ 19	+ 102	+ 163	+ 204	+ 229	+ 237
11-13............	255.0	722.5	— 1.500	— 4.25	+ 3.138	Z_1	0	— 185	— 370	— 554	— 739	— 924	— 1 109	— 1 043	— 978	— 913	— 848	— 783	— 717	— 652	— 587	— 522
						$Z = Z_0 + Z_1$	+ 4	— 74	— 157	— 243	— 329	— 422	— 521	— 378	— 248	— 130	— 28	+ 58	+ 124	+ 168	+ 196	+ 208
13-15............	297.5	680.0	— 1.598	— 3.643	+ 3.768	Z_1	0	— 158	— 317	— 476	— 634	— 792	— 950	— 1 109	— 1 039	— 970	— 901	— 831	— 762	— 693	— 624	— 554
						$Z = Z_0 + Z_1$	+ 4	— 47	— 104	— 165	— 224	— 290	— 362	— 444	— 309	— 187	— 81	+ 10	+ 79	+ 127	+ 159	+ 176
15-17............	340.0	637.5	— 1.700	— 3.188	+ 4.365	Z_1	0	— 139	— 277	— 416	— 554	— 693	— 832	— 970	— 1 109	— 1 035	— 961	— 887	— 813	— 739	— 665	— 591
						$Z = Z_0 + Z_1$	+ 4	— 28	— 64	— 105	— 144	— 191	— 244	— 305	— 379	— 252	— 141	— 46	+ 28	+ 81	+ 118	+ 139
17-19............	382.5	595.0	— 1.821	— 2.833	+ 4.883	Z_1	0	— 123	— 246	— 370	— 493	— 616	— 739	— 862	— 986	— 1 109	— 1 030	— 950	— 871	— 792	— 713	— 634
						$Z = Z_0 + Z_1$	+ 4	— 12	— 33	— 59	— 83	— 114	— 151	— 197	— 256	— 326	— 210	— 109	— 30	+ 28	+ 70	+ 96
19-21............	425.0	552.5	— 1.962	— 2.550	+ 5.270	Z_1	0	— 111	— 222	— 333	— 444	— 554	— 665	— 776	— 887	— 998	— 1 109	— 1 024	— 938	— 853	— 768	— 683
						$Z = Z_0 + Z_1$	+ 4	0	— 9	— 22	— 34	— 52	— 77	— 111	— 157	— 215	— 289	— 183	— 97	— 33	+ 15	+ 47
21-23............	467.5	510.0	— 2.125	— 2.319	+ 5.478	Z_1	0	— 101	— 202	— 302	— 403	— 504	— 605	— 706	— 806	— 907	— 1 008	— 1 109	— 1 016	— 924	— 831	— 739
						$Z = Z_0 + Z_1$	+ 4	+ 10	+ 11	+ 9	+ 7	— 2	— 17	— 41	— 76	— 124	— 188	— 268	— 175	— 104	— 48	— 9
						Z_1	0	— 96.6	— 193.2	— 289.8	— 386.4	— 483	— 579.5	— 676.1	— 772.7	— 869.3	— 965.9	— 1 062.5	— 1 062.5	— 965.9	— 869.3	— 772.7
23-23 A.........	488.75	488.75	— 2.222	— 2.222	+ 5.470	Correct Z_2	— 0.07	+ 0.1	+ 0.3	+ 0.6	+ 0.9	+ 1.3	+ 1.8	+ 2.5	+ 3.4	+ 4.5	+ 5.9	+ 7.5	+ 7.5	+ 5.9	+ 4.5	+ 3.4
						$Z = Z_0 + Z_1 + Z_2$	+ 4	+ 14	+ 20	+ 22	+ 24	+ 20	+ 10	— 9	— 39	— 82	— 140	— 214	— 214	— 140	— 82	— 39

ERMINATION OF INFLUENCE ORDINATES.—FOR BOTTOM CHORD. LIVE LOAD PER PANEL = 510 000 LB. = 1 000 P.

PANEL POINTS.—ALL VALUES IN THESE COLUMNS ARE INFLUENCE ORDINATES, AND ARE EXPRESSED IN THOUSANDTHS

2	4	6	8	10	12	14	16	18	20	22	22	20	18	16	14
111	+ 213	+ 311	+ 410	+ 502	+ 588	+ 665	+ 730	+ 788	+ 820	+ 841	+ 841	+ 820	+ 788	+ 730	+ 665
111	+ 213	+ 311	+ 410	+ 502	+ 588	+ 665	+ 730	+ 788	+ 820	+ 841	+ 841	+ 820	+ 788	+ 730	+ 665
272	− 259	− 247	− 235	− 222	− 210	− 198	− 185	− 173	− 161	− 143	− 136	− 124	− 111	− 99	− 86
161	− 46	+ 64	+ 175	+ 280	+ 378	+ 467	+ 545	+ 610	+ 659	+ 693	+ 705	+ 696	+ 672	+ 601	+ 579
288	− 476	− 454	− 431	− 408	− 386	− 363	− 340	− 318	− 395	− 272	− 250	− 227	− 204	− 182	− 159
127	− 263	− 148	− 21	+ 94	+ 202	+ 302	+ 390	+ 465	+ 525	+ 569	+ 591	+ 598	+ 579	+ 548	+ 506
205	− 411	− 616	− 585	− 554	− 524	− 493	− 463	− 431	− 400	− 370	− 339	− 308	− 277	− 246	− 216
91	− 198	− 305	− 175	− 52	+ 64	+ 172	+ 208	+ 352	+ 420	+ 471	+ 502	+ 512	+ 506	+ 484	+ 449
177	− 355	− 532	− 710	− 672	− 635	− 596	− 560	− 528	− 486	− 448	− 411	− 374	− 336	− 299	− 262
66	− 142	− 221	− 300	− 170	− 47	+ 67	+ 170	+ 260	+ 334	+ 393	+ 430	+ 446	+ 447	+ 431	+ 403
156	− 312	− 467	− 623	− 779	− 736	− 696	− 649	− 606	− 568	− 519	− 476	− 438	− 390	− 346	− 303
45	− 99	− 156	− 213	− 277	− 148	− 28	+ 81	+ 177	+ 257	+ 332	+ 365	+ 387	+ 393	+ 384	+ 362
139	− 277	− 416	− 554	− 692	− 831	− 782	− 733	− 684	− 635	− 587	− 538	− 489	− 440	− 391	− 343
28	− 64	− 105	− 144	− 190	− 248	− 117	− 3	+ 99	+ 185	+ 254	+ 308	+ 331	+ 348	+ 339	+ 323
124	− 249	− 378	− 497	− 621	− 746	− 870	− 815	− 761	− 707	− 652	− 598	− 544	− 489	− 435	− 381
13	− 35	− 62	− 87	− 119	− 168	− 205	− 85	+ 22	+ 118	+ 169	+ 243	+ 276	+ 294	+ 295	+ 284
112	− 225	− 337	− 449	− 561	− 674	− 786	− 898	− 838	− 779	− 719	− 659	− 599	− 539	− 479	− 419
1	− 12	− 26	− 39	− 59	− 86	− 121	− 168	− 55	+ 41	+ 122	+ 182	+ 221	+ 244	+ 251	+ 246
102	− 204	− 306	− 408	− 510	− 612	− 714	− 816	− 918	− 858	− 787	− 722	− 656	− 590	− 525	− 459
9	+ 9	+ 5	+ 2	− 8	− 24	− 49	− 86	− 185	− 88	+ 54	+ 119	+ 164	+ 193	+ 205	+ 206
98	− 186	− 279	− 372	− 466	− 559	− 652	− 745	− 838	− 981	− 839	− 788	− 716	− 645	− 573	− 501
18	+ 27	+ 32	+ 38	+ 35	+ 29	+ 13	− 15	− 55	− 111	− 18	+ 58	+ 104	+ 138	+ 157	+ 164
82	− 163	− 245	− 327	− 408	− 490	− 572	− 653	− 785	− 817	− 896	− 898	− 817	− 725	− 653	− 572
0.07	− 0.27	− 0.52	− 0.75	− 1.10	− 1.55	− 2.12	− 2.86	− 3.81	− 4.97	− 6.37	− 6.37	− 4.97	− 3.81	− 0.86	− 2.12
29	+ 50	+ 65	+ 82	+ 93	+ 96	+ 91	+ 74	+ 44	− 2	− 63	− 63	− 2	+ 44	+ 74	+ 91

DETERMINATION OF INFLUENCE ORDINATES.—FOR TOP CHORD. LIVE LOAD PER PANEL = 510 000 LB. = 1 000 P. AL

PANEL POINTS.—ALL VALUES IN THESE COLUMNS ARE INFLUENCE ORDINATES, AND ARE EXPRESSED IN THOUSANDTH

2	4	6	8	10	12	14	16	18	20	22	22	20	18	16	14
111	+ 218	+ 311	+ 410	+ 503	+ 588	+ 665	+ 730	+ 788	+ 820	+ 841	+ 841	+ 820	+ 788	+ 730	+ 665
1 109	− 1 059	− 1 008	− 958	− 907	− 557	− 807	− 756	− 706	− 655	− 605	− 555	− 504	− 454	− 403	− 353
998	− 846	− 697	− 548	− 405	− 269	− 142	− 25	+ 77	+ 165	+ 236	+ 286	+ 316	+ 329	+ 327	+ 312
554	− 1 109	− 1 056	− 1 008	− 950	− 898	− 845	− 793	− 739	− 696	− 634	− 581	− 528	− 475	− 422	− 370
443	− 896	− 745	− 598	− 448	− 310	− 180	− 62	+ 44	+ 134	+ 207	+ 260	+ 293	+ 308	+ 308	+ 295
870	− 739	− 1 109	− 1 058	− 998	− 942	− 887	− 832	− 776	− 721	− 665	− 610	− 554	− 499	− 444	− 388
259	− 526	− 798	− 643	− 496	− 354	− 222	− 109	+ 7	+ 99	+ 176	+ 231	+ 266	+ 284	+ 286	+ 277
277	− 554	− 832	− 1 109	− 1 050	− 992	− 934	− 875	− 816	− 759	− 700	− 642	− 584	− 525	− 467	− 408
166	− 341	− 521	− 699	− 548	− 404	− 269	− 145	− 34	+ 61	+ 141	+ 199	+ 236	+ 258	+ 263	+ 257
223	− 448	− 665	− 887	− 1 109	− 1 047	− 986	− 924	− 862	− 801	− 739	− 678	− 616	− 554	− 493	− 431
111	− 230	− 354	− 477	− 607	− 459	− 321	− 194	− 79	+ 19	+ 102	+ 168	+ 204	+ 229	+ 237	+ 234
185	− 370	− 554	− 789	− 924	− 1 100	− 1 043	− 978	− 913	− 848	− 783	− 717	− 658	− 587	− 522	− 456
74	− 157	− 243	− 329	− 423	− 521	− 378	− 246	− 130	− 28	+ 58	+ 124	+ 168	+ 196	+ 208	+ 209
158	− 317	− 476	− 634	− 792	− 950	− 1 109	− 1 039	− 970	− 901	− 831	− 762	− 696	− 624	− 554	− 485
47	− 104	− 165	− 224	− 290	− 362	− 444	− 309	− 187	− 81	+ 10	+ 79	+ 127	+ 159	+ 176	+ 180
139	− 277	− 416	− 554	− 693	− 832	− 970	− 1 109	− 1 035	− 961	− 887	− 813	− 739	− 665	− 591	− 517
28	− 64	− 105	− 144	− 191	− 244	− 306	− 379	− 252	− 141	− 46	+ 28	+ 81	+ 118	+ 139	+ 148
123	− 246	− 370	− 493	− 616	− 739	− 863	− 986	− 1 109	− 1 030	− 950	− 871	− 792	− 713	− 634	− 554
12	− 33	− 59	− 83	− 114	− 151	− 197	− 256	− 326	− 210	− 109	− 30	+ 28	+ 70	+ 96	+ 111
111	− 222	− 333	− 444	− 554	− 665	− 776	− 887	− 998	− 1 109	− 1 024	− 938	− 853	− 768	− 683	− 597
0	− 9	− 22	− 34	− 53	− 77	− 111	− 157	− 215	− 280	− 163	− 97	− 38	+ 15	+ 47	+ 68
101	− 202	− 303	− 403	− 504	− 605	− 706	− 806	− 907	− 1 008	− 1 109	− 1 016	− 924	− 833	− 759	− 647
10	+ 11	+ 9	+ 7	− 2	− 17	− 41	− 76	− 124	− 156	− 268	− 175	− 104	− 49	− 9	+ 18
96.6	− 193.2	− 289.8	− 386.4	− 483	− 579.5	− 676.1	− 772.7	− 860.3	− 965.9	− 1 062.5	− 1 062.5	− 965.9	− 860.3	− 772.7	− 676.1
0.1	+ 0.3	+ 0.6	+ 0.9	+ 1.3	+ 1.8	+ 2.5	+ 3.4	+ 4.5	+ 5.9	+ 7.5	+ 7.5	+ 5.9	+ 4.5	+ 3.4	+ 2.5
14	+ 20	+ 22	+ 24	+ 20	+ 10	− 9	− 39	− 82	− 140	− 211	− 214	− 140	− 82	− 39	− 9

P. ALL STRESSES GIVEN IN UNITS OF 1 000 LB.

+ Denotes tension.
− Denotes compression.

…ANDTHS OF A UNIT.								(31) INFLUENCE AREA, $Z = \Sigma z$.	(32)	(33)	(34) LIVE-LOAD STRESS $= P \frac{y}{r} \Sigma z$.	(35)
4	12	10	8	6	4	2	0	Positive. +	Negative. −	Total.	+	−
35	+ 588	+ 502	+ 410	+ 311	+ 218	+ 111	+ 4					
35	+ 588	+ 502	+ 410	+ 311	+ 213	+ 111	+ 4	11.969	0.	+ 11.969	……	8 057
36	− 74	− 63	− 49	− 37	− 25	− 12	0	9.021	0.192	+ 8.829	165	7 746
79	+ 514	+ 440	+ 361	+ 274	+ 188	+ 99	+ 4					
59	− 186	− 113	− 91	− 68	− 45	− 23	0	7.019	0.543	+ 6.476	594	7 692
06	+ 452	+ 389	+ 319	+ 243	+ 168	+ 88	+ 4					
16	− 185	− 154	− 128	− 92	− 60	− 31	0	5.678	0.808	+ 4.870	1 095	7 692
49	+ 408	+ 348	+ 287	+ 219	+ 151	+ 80	+ 4					
52	− 224	− 187	− 149	− 112	− 75	− 37	0	4.722	0.930	+ 3.792	1 504	7 688
08	+ 364	+ 315	+ 261	+ 199	+ 138	+ 74	+ 4					
19	− 260	− 216	− 173	− 129	− 86	− 43	0	3.950	0.954	+ 2.996	1 809	7 492
32	+ 328	+ 286	+ 237	+ 182	+ 127	+ 68	+ 4					
43	− 298	− 244	− 196	− 147	− 96	− 49	0	3.288	0.891	+ 2.397	1 949	7 192
23	+ 295	+ 258	+ 214	+ 164	+ 115	+ 63	+ 4					
31	− 326	− 272	− 217	− 163	− 109	− 54	0	2.706	0.755	+ 1.951	1 878	6 712
34	+ 262	+ 230	+ 198	+ 148	+ 104	+ 57	+ 4					
19	− 359	− 300	− 240	− 180	− 120	− 60	0	2.178	0.555	+ 1.623	1 582	6 018
46	+ 229	+ 203	+ 170	+ 131	+ 93	+ 51	+ 4					
59	− 394	− 328	− 263	− 197	− 131	− 66	0	1.718	0.324	+ 1.394	974	5 165
06	+ 194	+ 174	+ 148	+ 114	+ 82	+ 45	+ 4					
01	− 430	− 358	− 287	− 215	− 143	− 72	0	1.438	0.188	+ 1.250	600	4 591
34	+ 158	+ 144	+ 128	+ 96	+ 70	+ 39	+ 4					
72	− 490	− 408	− 327	− 245	− 168	− 82	0					
2.12	− 1.55	− 1.10	− 0.75	− 0.52	− 0.27	− 0.07	+ 0.06	1.252	0.128	+ 1.124	422	4 131
91	+ 96	+ 93	+ 83	+ 65	+ 50	+ 29	+ 5					

Stress 22–22A $= -6.47\,v_H + 0.5223\,m - v_S \cdot \cos.\beta$
$= -6.47\,(v_H - 0.08165\,m + 0.1123\,v_S)$
$= -6.47\,(Z_0 + Z_1 + Z_2)$.

ALL STRESSES GIVEN IN UNITS OF 1 000 LB.

+ Denotes tension.
− Denotes compression.

…ANDTHS OF A UNIT.								(31) INFLUENCE AREA, $Z = \Sigma z$.	(32)	(33)	(34) LIVE-LOAD STRESS $= P \frac{y}{r} \Sigma z$.	(35)
14	12	10	8	6	4	2	0	Positive. +	Negative. −	Total.	+	−
665	+ 588	+ 502	+ 410	+ 311	+ 218	+ 111	+ 4	……	……	……	……	……
358	− 303	− 252	− 202	− 151	− 101	− 50	0	3.119	3.916	− 0.797	530	665
312	+ 286	+ 260	+ 208	+ 160	+ 112	+ 61	+ 4					
370	− 317	− 264	− 211	− 158	− 106	− 53	0	2.865	3.662	− 0.797	1 158	1 474
295	+ 271	+ 238	+ 190	+ 153	+ 107	+ 58	+ 4					
388	− 333	− 277	− 223	− 166	− 111	− 55	0	2.598	3.395	− 0.797	1 785	2 332
277	+ 255	+ 225	+ 183	+ 145	+ 102	+ 56	+ 4					
408	− 350	− 292	− 234	− 175	− 117	− 58	0	2.317	3.114	− 0.797	2 261	3 066
257	+ 238	+ 210	+ 176	+ 136	+ 96	+ 53	+ 4					
431	− 370	− 308	− 246	− 185	− 123	− 62	0	2.025	2.822	− 0.797	2 602	3 626
234	+ 216	+ 194	+ 154	+ 126	+ 90	+ 49	+ 4					
456	− 391	− 326	− 261	− 196	− 130	− 65	0	1.724	2.519	− 0.797	2 759	4 081
209	+ 197	+ 176	+ 149	+ 115	+ 83	+ 46	+ 4					
485	− 416	− 347	− 277	− 206	− 139	− 69	0	1.410	2.207	− 0.797	2 706	4 236
180	+ 172	+ 155	+ 133	+ 103	+ 74	+ 42	+ 4					
517	− 443	− 370	− 296	− 222	− 148	− 74	0	1.092	1.889	− 0.797	2 429	4 202
148	+ 145	+ 132	+ 114	+ 89	+ 65	+ 37	+ 4					
554	− 475	− 396	− 317	− 238	− 158	− 79	0	0.774	1.571	− 0.797	1 997	3 912
111	+ 113	+ 106	+ 93	+ 73	+ 55	+ 32	+ 4					
597	− 512	− 427	− 341	− 256	− 171	− 85	0	0.475	1.272	− 0.797	1 277	3 419
68	+ 76	+ 75	+ 69	+ 55	+ 42	+ 26	+ 4					
647	− 554	− 462	− 370	− 277	− 185	− 92	0	0.252	1.049	− 0.797	704	2 931
18	+ 34	+ 40	+ 40	+ 34	+ 28	+ 19	+ 4					
676.1	− 579.5	− 483	− 386.4	− 289.8	− 198.2	− 96.6	0					
2.5	+ 1.8	+ 1.8	+ 0.9	+ 0.6	+ 0.3	+ 0.1	− 0.07	0.221	0.963	− 0.742	617	2 686
9	+ 10	+ 20	+ 24	+ 22	+ 20	+ 14	+ 4					

Stress 25–25A $= +5.47\,(v_H - 0.0666\,m - 0.1266\,v_S)$
$= +5.47\,(Z_0 + Z_1 + Z_2)$.

TABLE 15.—DETERMI

Member.	(1) X, in feet.	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0	2	4
Influence ordinate. Hor. reaction.....	....	....	....	....	....	Z_0	+ 4	+ 111	+ 213
1-2...........	219.83	757.67	− 1.159	− 3.995	− 0.854	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 1 109 − 998	− 1 058 − 845
3-4...........	262.45	695.05	− 1.256	− 3.069	− 0.910	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 134 − 23	− 1 147 − 934
5-6...........	390.45	587.05	− 1.367	− 2.056	− 0.868	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 89 + 22	− 179 + 34
7-8...........	512.24	465.26	− 1.500	− 1.363	− 0.850	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 59 + 52	− 118 + 95
9-10...........	561.12	416.38	− 1.643	− 1.220	− 0.860	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 53 + 58	− 106 + 107
11-12...........	619.10	358.40	− 1.814	− 1.050	− 0.898	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 46 + 65	− 91 + 122
13-14...........	691.75	285.75	− 2.027	− 0.837	− 0.901	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 36 + 75	− 73 + 140
15-16...........	790.20	187.30	− 2.316	− 0.549	− 0.842	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 24 + 87	− 48 + 165
17-18...........	941.55	35.95	− 2.757	− 0.105	− 0.734	Z_1 $Z = Z_0 + Z_1$	0 + 4	− 5 + 106	− 9 + 204
19-20...........	1 220.89	243.39	− 3.575	+ 0.713	− 0.554	Z_1 $Z = Z_0 + Z_1$	0 + 4	+ 31 + 142	+ 62 + 275
21-22...........	2 008.46	1 025.58	− 5.884	+ 3.012	− 0.305	Z_1 $Z = Z_0 + Z_1$	0 + 4	+ 133 + 244	+ 262 + 475
23-22...........	∞	....	....	....	....	Z_1 Z_2 $Z = Z_1 + Z_2$	0 − 0.5 − 0.5	+ 31.63 + 0.6 + 32	+ 63. + 2. + 66

TABLE 16.—

Member.	(1) X, in feet.	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0
Influence ordinate, Hor. reaction........	....	....	....	....	....	Z_0	+ 4
0-1...........	42.50	985.00	− 1.159	− 25.500	0.988 Above floor. Below floor.	Z_1 $Z = Z_0 + Z_1$ $Z = Z_0 + Z_1$	0 + 4 − 1 155
2-3...........	244.74	782.76	− 1.258	− 3.758	+ 0.986	Z_1 $Z = Z_0 + Z_1$ $Z = Z_0 + Z_1$	0 + 4
4-5...........	321.96	655.52	− 1.357	− 2.763	+ 1.001	Z_1 $Z = Z_0 + Z_1$ $Z = Z_0 + Z_1$	0 + 4
6-7...........	481.77	495.73	− 1.495	− 1.538	+ 0.910	Z_1 $Z = Z_0 + Z_1$	0 + 4
8-9...........	719.61	257.89	− 1.680	− 0.602	+ 0.780	Z_1 $Z = Z_0 + Z_1$	0 + 4
10-11...........	825.18	152.32	− 1.883	− 0.348	+ 0.715	Z_1 $Z = Z_0 + Z_1$	0 + 4
12-13...........	967.16	9.66	− 2.100	+ 0.021	+ 0.684	Z_1 $Z = Z_0 + Z_1$	0 + 4
14-15...........	1 291.22	313.72	− 2.591	+ 0.629	+ 0.502	Z_1 $Z = Z_0 + Z_1$	0 + 4
16-17...........	2 166.72	1 189.22	− 3.440	+ 1.888	+ 0.345	Z_1 $Z = Z_0 + Z_1$	0 + 4
18-19...........	80 066	81 043	− 6.452	+ 6.530	+ 0.154	Z_1 $Z = Z_0 + Z_1$	0 + 4
20-21...........	1 131.54	2 109.04	+ 11.964	− 22.298	− 0.061	Z_1 $Z = Z_0 + Z_1$	0 + 4
22-23...........	302.21	1 279.71	+ 1.374	− 5.817	− 0.286	Z_1 Z_2 $Z = Z_0 + Z_1 + Z_2$	0 + 1.9 + 5

…ermination of Influence Ordinates.—For Diagonals. Live Load per Panel = 510 000 lb.

Panel Points.—All Values in These Columns are Influence Ordinates, and are …

4	6	8	10	12	14	16	18	20	22	24
218	+ 311	+ …10	+ 508	+ 588	+ 665	+ 730	+ 788	+ 829	+ 841	+ 841
1 058	− 1 008	− …98	− 907	− 856	− 806	− 756	− 706	− 655	− 605	− 554
845	− 697	− …48	− 405	− 268	− 141	− 26	+ 77	+ 165	+ 236	+ 227
1 147	− 1 002	− 1 …88	− 953	− 929	− 874	− 819	− 765	− 710	− 655	− 601
984	− 781	− …38	− 481	− 341	− 209	− 89	+ 18	+ 110	+ 186	+ 249
179	− 1 189	− 1 …29	− 1 070	− 1 011	− 951	− 898	− 832	− 773	− 713	− 684
84	− 878	− …19	− 568	− 423	− 286	− 163	− 49	+ 47	+ 128	+ 187
118	− 178	− 1 …39	− 1 174	− 1 109	− 1 044	− 979	− 913	− 848	− 783	− 718
95	+ 183	− …29	− 672	− 521	− 379	− 249	− 130	− 28	+ 58	+ 123
106	− 159	− …12	− 1 286	− 1 215	− 1 144	− 1 072	− 1 001	− 930	− 857	− 786
107	+ 158	+ …98	− 784	− 627	+ 479	− 342	− 216	− 106	− 18	+ 55
91	− 187	− …83	− 298	− 1 341	− 1 252	− 1 183	− 1 104	− 1 035	− 965	− 897
122	+ 174	+ …27	+ 274	− 738	− 597	− 458	− 321	− 205	− 105	− 20
78	− 109	− …46	− 183	− 218	− 1 410	− 1 332	− 1 264	− 1 146	− 1 058	− 989
140	+ 202	+ …64	+ 320	+ 370	− 745	− 592	− 451	− 326	− 217	− 128
48	− 73	− …95	− 119	− 148	− 167	− 1 510	− 1 410	− 1 309	− 1 208	− 1 108
165	+ 289	+ …15	+ 388	+ 445	+ 498	− 780	− 627	− 489	− 367	− 267
9	− 14	− …18	− 23	− 27	− 32	− 37	− 1 678	− 1 568	− 1 436	− 1 318
204	+ 297	+ …82	+ 479	+ 561	+ 638	+ 698	− 805	− 738	− 597	− 477
68	+ 93	+ …94	+ 155	+ 186	+ 217	+ 248	+ 279	− 2 081	− 1 865	− 1 710
275	+ 404	+ …34	+ 687	+ 774	+ 852	+ 978	+ 1 089	− 1 201	− 1 624	− 869
268	+ 398	+ …94	+ 655	+ 786	+ 917	+ 1 048	+ 1 179	+ 1 310	− 2 070	− 2 814
475	+ 704	+ …84	+ 1 157	+ 1 374	+ 1 583	+ 1 778	+ 1 968	+ 2 180	− 2 229	− 1 973
68.25	+ 94.86	+ …6.51	+ 158.14	+ 189.76	+ 221.39	+ 253.02	+ 284.64	+ 316.27	+ 347.90	− 347.90
2.4	+ 4.6	+ …6.7	+ 9.8	+ 18.8	+ 18.9	+ 25.5	+ 33.9	+ 44.8	+ 56.7	+ 56.7
66	+ 99	+ …33	+ 168	+ 204	+ 240	+ 279	+ 319	+ 361	+ 405	− 291

16.—Determination of Influence Ordinates.—For Verticals. Live Load per Panel = 510 00…

Panel Points.—All Values in These Columns Are Influence Ordinates and Are …

2	4	6	8	10	12	14	16	18	20	22	24	26
+ 111	+ 213	+ 311	+ 410	+ 508	+ 588	+ 665	+ 730	+ 788	+ 830	+ 841	+ 841	+ 820
− 1 109	− 1 066	− 1 008	− 958	− 907	− 857	− 806	− 756	− 706	− 655	− 605	− 554	− 504
− 996	− 945	− 697	− 546	− 405	− 269	− 141	− 26	+ 77	+ 165	+ 236	+ 297	+ 316
− 168	− 1 144	− 1 090	− 1 085	− 951	− 926	− 873	− 817	− 763	− 708	− 654	− 599	− 545
− 52 − 1 088	− 981	− 779	− 665	− 479	− 338	− 207	− 87	+ 90	+ 119	+ 197	+ 242	+ 275
− 120	− 240	− 180	− 1 121	− 1 062	− 1 008	− 944	− 886	− 826	− 767	− 709	− 649	− 590
− 9	− 27 − 1 026	− 869	− 711	− 560	− 415	− 279	− 155	− 43	+ 53	+ 139	+ 192	+ 290
− 67	− 133	− 201	− 1 225	− 1 170	− 1 105	− 1 040	− 975	− 910	− 845	− 780	− 715	− 650
+ 41	+ 80	+ 110	− 895	− 668	− 517	− 375	− 245	− 127	− 25	+ 61	+ 125	+ 170
− 26	− 52	− 79	− 105	− 1 315	− 1 249	− 1 169	− 1 096	− 1 022	− 949	− 877	− 804	− 730
+ 85	+ 161	+ 232	+ 305	− 818	− 654	− 504	− 365	− 239	− 129	− 35	+ 37	+ 90
− 15	− 30	− 45	− 60	− 76	− 1 392	− 1 310	− 1 228	− 1 146	− 1 064	− 982	− 900	− 819
+ 96	+ 182	+ 266	+ 350	+ 436	− 804	− 645	− 498	− 363	− 244	− 141	− 59	+ 1
+ 1	+ 2	+ 3	+ 4	+ 5	+ 6	− 1 502	− 1 408	− 1 314	− 1 221	− 1 127	− 1 032	− 939
+ 112	+ 215	+ 314	+ 414	+ 507	+ 594	− 837	− 678	− 531	− 401	− 276	− 193	− 119
+ 27	+ 55	+ 82	+ 109	+ 137	+ 164	+ 192	− 1 690	− 1 577	− 1 464	− 1 358	− 1 239	− 1 126
+ 183	+ 268	+ 298	+ 519	+ 679	+ 759	+ 357	− 960	− 794	− 634	− 511	− 328	− 306
+ 63	+ 164	+ 246	+ 328	+ 410	+ 498	+ 575	+ 657	− 2 094	− 1 944	− 1 795	− 1 645	− 1 496
+ 198	+ 377	+ 537	+ 738	+ 912	+ 1 061	+ 1 240	+ 1 387	− 1 311	− 1 124	− 954	− 834	− 675
+ 284	+ 568	+ 852	+ 1 136	+ 1 420	+ 1 704	+ 1 988	+ 2 271	+ 2 555	− 2 647	− 3 246	− 3 065	− 2 905
+ 395	+ 751	+ 153	+ 1 546	+ 1 922	+ 2 292	+ 2 653	+ 3 001	+ 3 338	− 2 327	− 2 325	− 2 145	− 1 965
− 960	− 1 994	− 2 909	− 3 878	− 4 848	− 5 817	− 6 787	− 7 756	− 8 725	− 9 695	+ 6 948	+ 5 721	+ 5 201
− 838	− 1 726	− 2 508	− 3 468	− 4 346	− 5 229	− 6 122	− 7 085	− 7 942	− 8 875	+ 7 088	+ 6 563	+ 6 081
− 177	− 354	− 531	− 707	− 894	− 1 061	− 1 238	− 1 415	− 1 592	− 1 765	− 1 946	− 179	− 158
− 1.4	− 5.8	− 11.1	− 16.1	− 22.5	− 33.2	− 45.4	− 61.8	− 81.5	− 106.7	− 130.4	− 130.4	− 106.7
− 67	− 147	− 231	− 313	− 406	− 505	− 616	− 746	− 891	− 1 055	− 1 240	+ 585	+ 550

NCE ORDINATES.—FOR DIAGONALS. LIVE LOAD PER PANEL = 510 000 LB. = 1 000 P. ALL STRESSES GIVEN IN U

PANEL POINTS.—ALL VALUES IN THESE COLUMNS ARE INFLUENCE ORDINATES, AND ARE EXPRESSED IN THOUSANDTHS OF A UNIT.

10	12	14	16	18	20	22	22	20	18	16	14	12
+ 502	+ 588	+ 665	+ 730	+ 783	+ 820	+ 841	+ 841	+ 820	+ 783	+ 730	+ 665	+ 588
− 907	− 856	− 806	− 756	− 706	− 655	− 605	− 554	− 504	− 455	− 408	− 353	− 302
− 405	− 268	− 141	− 26	+ 77	+ 165	+ 236	+ 287	+ 316	+ 328	+ 327	+ 312	+ 286
− 983	− 929	− 874	− 819	− 765	− 710	− 655	− 601	− 546	− 493	− 457	− 392	− 328
− 481	− 341	− 209	− 69	+ 18	+ 110	+ 186	+ 240	+ 274	+ 291	+ 298	+ 283	+ 260
− 1 070	− 1 011	− 951	− 898	− 822	− 773	− 713	− 654	− 594	− 535	− 476	− 416	− 357
− 568	− 423	− 286	− 163	− 49	+ 47	+ 128	+ 187	+ 226	+ 248	+ 264	+ 249	+ 231
− 1 174	− 1 109	− 1 044	− 979	− 918	− 848	− 783	− 718	− 652	− 587	− 522	− 457	− 392
− 672	− 521	− 379	− 249	− 180	− 28	+ 58	+ 123	+ 168	+ 196	+ 208	+ 208	+ 196
− 1 286	− 1 215	− 1 144	− 1 072	− 1 001	− 929	− 857	− 786	− 715	− 643	− 576	− 500	− 429
− 784	− 627	+ 479	− 342	− 218	− 109	− 16	+ 55	+ 105	+ 140	+ 158	+ 165	+ 159
− 298	− 1 341	− 1 282	− 1 188	− 1 104	− 1 025	− 946	− 867	− 789	− 710	− 631	− 552	− 473
+ 274	− 753	− 597	− 456	− 321	− 205	− 105	− 26	+ 31	+ 73	+ 99	+ 113	+ 115
− 188	− 216	− 1 410	− 1 322	− 1 234	− 1 146	− 1 058	− 969	− 881	− 793	− 705	− 617	− 529
+ 820	+ 870	− 745	− 592	− 451	− 326	− 217	− 128	− 61	− 10	+ 85	+ 48	+ 59
− 119	− 148	− 167	− 1 510	− 1 410	− 1 309	− 1 208	− 1 108	− 1 007	− 907	− 806	− 705	− 604
+ 383	+ 445	+ 496	− 780	− 627	− 469	− 367	− 267	− 167	− 124	− 76	− 40	− 16
− 23	− 27	− 32	− 37	− 1 678	− 1 556	− 1 438	− 1 318	− 1 199	− 1 079	− 959	− 839	− 719
+ 479	+ 561	+ 633	+ 698	− 895	− 788	− 597	− 477	− 379	− 296	− 229	− 174	− 131
+ 155	+ 186	+ 217	+ 248	+ 279	− 2 021	− 1 865	− 1 710	− 1 554	− 1 390	− 1 244	− 1 068	− 933
+ 657	+ 774	+ 882	+ 978	+ 1 062	− 1 201	− 1 024	− 869	− 784	− 616	− 514	− 428	− 345
+ 655	+ 786	+ 917	+ 1 048	+ 1 179	+ 1 310	− 3 070	− 2 814	− 2 559	− 2 303	− 2 047	− 1 791	− 1 535
+ 115.7	+ 1 374	+ 1 582	+ 1 778	+ 1 962	+ 2 180	− 2 229	− 1 973	− 1 739	− 1 520	− 1 317	− 1 126	− 947
+ 158.14	+ 189.76	+ 221.39	+ 258.02	+ 284.64	+ 316.27	+ 347.90	− 347.90	− 316.27	− 284.64	− 258.02	− 221.39	− 189.76
+ 9.8	+ 13.8	+ 18.9	+ 25.5	+ 33.9	+ 44.3	+ 56.7	+ 56.7	+ 44.3	+ 33.9	+ 25.5	+ 18.9	+ 13.8
+ 166	+ 204	+ 240	+ 279	+ 319	+ 361	+ 405	− 291	− 272	− 251	− 226	− 208	− 176

INFLUENCE ORDINATES.—FOR VERTICALS. LIVE LOAD PER PANEL = 510 000 LB. = 1 000 P. ALL STRESSES GIVEN

PANEL POINTS.—ALL VALUES IN THESE COLUMNS ARE INFLUENCE ORDINATES AND ARE EXPRESSED IN THOUSANDTHS OF A UNIT.

8	10	12	14	16	18	20	22	22	20	18	16	14	12	10	8
+ 410	+ 502	+ 588	+ 665	+ 730	+ 783	+ 820	+ 841	+ 841	+ 820	+ 783	+ 730	+ 665	+ 588	+ 502	+ [illegible]
− 958	− 907	− 857	− 806	− 756	− 706	− 655	− 605	− 554	− 504	− 454	− 403	− 353	− 302	− 252	− [illegible]
− 548	− 405	− 269	− 141	− 26	+ 77	+ 165	+ 236	+ 287	+ 316	+ 329	+ 327	+ 312	+ 286	+ 250	+ [illegible]
− 1 085	− 981	− 926	− 872	− 817	− 763	− 708	− 654	− 599	− 545	− 490	− 436	− 382	− 327	− 273	− [illegible]
− 625	− 479	− 338	− 207	− 87	+ 20	+ 112	+ 187	+ 242	+ 275	+ 293	+ 294	+ 283	+ 261	+ 220	+ [illegible]
− 1 121	− 1 062	− 1 008	− 944	− 885	− 826	− 767	− 709	− 649	− 590	− 531	− 472	− 413	− 354	− 295	− [illegible]
− 711	− 560	− 415	− 279	− 155	− 43	+ 58	+ 132	+ 192	+ 230	+ 252	+ 258	+ 253	+ 234	+ 207	+ [illegible]
− 1 235	− 1 170	− 1 105	− 1 040	− 975	− 910	− 845	− 780	− 715	− 650	− 585	− 520	− 455	− 390	− 325	− [illegible]
− 825	− 668	− 517	− 375	− 245	− 127	− 25	+ 61	+ 126	+ 170	+ 198	+ 210	+ 210	+ 198	+ 177	+ [illegible]
− 105	− 1 315	− 1 242	− 1 169	− 1 096	− 1 022	− 949	− 877	− 804	− 730	− 657	− 584	− 511	− 438	− 365	− [illegible]
+ 305	− 813	− 654	− 504	− 366	− 239	− 129	− 36	+ 37	+ 90	+ 126	+ 146	+ 154	+ 150	+ 137	+ [illegible]
− 60	− 76	− 1 392	− 1 310	− 1 228	− 1 146	− 1 064	− 983	− 900	− 819	− 737	− 655	− 578	− 491	− 409	− [illegible]
+ 350	+ 496	− 804	− 645	− 498	− 363	− 244	− 141	− 59	+ 1	+ 46	+ 75	+ 93	+ 97	+ 93	+ [illegible]
+ 4	+ 5	+ 6	− 1 502	− 1 408	− 1 314	− 1 221	− 1 127	− 1 033	− 939	− 845	− 751	− 657	− 562	− 470	− [illegible]
+ 414	+ 507	+ 594	− 657	− 678	− 531	− 401	− 286	− 192	− 119	− 63	− 21	+ 8	+ 25	+ 33	+ [illegible]
+ 109	+ 187	+ 164	+ 192	− 1 690	− 1 577	− 1 464	− 1 352	− 1 239	− 1 126	− 1 014	− 901	− 788	− 676	− 563	− [illegible]
+ 519	+ 639	+ 752	+ 857	− 960	− 794	− 644	− 511	− 398	− 306	− 231	− 171	− 123	− 38	− 61	− [illegible]
+ 828	+ 410	+ 493	+ 575	+ 657	− 2 094	− 1 944	− 1 795	− 1 645	− 1 496	− 1 346	− 1 197	− 1 047	− 897	− 746	− [illegible]
+ 738	+ 912	+ 1 081	+ 1 240	+ 1 387	− 1 311	− 1 124	− 954	− 804	− 676	− 563	− 467	− 382	− 309	− 246	− [illegible]
+ 1 136	+ 1 420	+ 1 704	+ 1 988	+ 2 271	+ 2 555	− 3 647	− 3 366	− 3 086	− 2 805	− 2 525	− 2 244	− 1 954	− 1 653	− 1 403	− 1 [illegible]
+ 1 546	+ 1 922	+ 2 292	+ 2 653	+ 3 001	+ 3 338	− 2 827	− 2 525	− 2 245	− 1 985	− 1 742	− 1 514	− 1 310	− 1 095	− 901	− [illegible]
− 3 878	− 4 848	− 5 817	− 6 787	− 7 756	− 8 725	− 9 695	+ 6 242	+ 5 721	+ 5 201	+ 4 681	+ 4 161	+ 3 663	+ 3 121	+ 2 600	+ 2 [illegible]
− 3 468	− 4 346	− 5 229	− 6 122	− 7 026	− 7 942	− 8 875	+ 7 083	+ 6 562	+ 6 021	+ 5 461	+ 4 891	+ 4 323	+ 3 709	+ 3 102	+ [illegible]
− 707	− 824	− 1 061	− 1 238	− 1 415	− 1 592	− 1 768	− 1 945	− 179	− 163	− 147.0	− 130	− 114	− 984	− 82	− [illegible]
− 16.1	− 23.6	− 33.2	− 45.4	− 61.8	− 81.5	− 106.7	− 136.4	− 136.4	− 106.7	− 81.5	− 61.8	− 45.4	− 33.2	− 23.6	− [illegible]
− 318	− 406	− 506	− 618	− 746	− 891	− 1 055	− 1 240	+ 696	+ 550	+ 554	+ 539	+ 506	+ 457	+ 396	+ [illegible]

…IN UNITS OF 1 000 LB.

+ Denotes tension.
− Denotes compression.

…2	10	8	6	4	2	0	(31) Influence, Area $Z = \Sigma z$. Positive. +	(32) Negative. −	(33) Total.	(34) Live-Load Stress $= P\frac{y}{r}\Sigma z$. +	(35) −
…388	+ 508	+ 410	+ 311	+ 213	+ 111	+ 4					
…302	− 252	− 202	− 151	− 101	− 50	0	3.119	3.916	− 0.797	1 705	1 356
…296	+ 250	+ 208	+ 160	+ 112	+ 61	+ 4					
…228	− 273	− 218	− 164	− 109	− 55	0	2.680	3.477	− 0.797	1 618	1 249
…260	+ 289	+ 198	+ 147	+ 104	+ 56	+ 4					
…257	− 297	− 296	− 178	− 119	− 59	0	2.260	3.057	− 0.797	1 384	1 068
…281	+ 205	+ 172	+ 128	+ 94	+ 52	+ 4					
…392	− 327	− 261	− 196	− 130	− 65	0	1.944	2.741	− 0.797	1 189	848
…196	+ 175	+ 149	+ 115	+ 83	+ 46	+ 4					
…129	− 357	− 266	− 215	− 142	− 72	0	1.692	2.489	− 0.797	1 118	760
…159	+ 145	+ 124	+ 96	+ 70	+ 39	+ 4					
…478	− 394	− 315	− 237	− 158	− 79	0	1.555	2.352	− 0.797	1 077	712
…115	+ 108	+ 95	+ 74	+ 55	+ 32	+ 4					
…529	− 441	− 353	− 265	− 176	− 88	0	1.606	2.403	− 0.797	1 098	730
…59	+ 61	+ 57	+ 46	+ 37	+ 23	+ 4					
…304	− 506	− 408	− 302	− 201	− 101	0	2.084	2.881	− 0.797	1 212	870
…16	− 1	+ 7	+ 9	+ 12	+ 10	+ 4					
…719	− 599	− 460	− 360	− 240	− 120	0	3.174	3.971	− 0.797	1 465	1 187
…131	− 97	− 70	− 49	− 27	− 9	+ 4					
…088	− 777	− 622	− 467	− 311	− 165	0	5.431	6.228	− 0.797	1 762	1 536
…045	− 275	− 212	− 156	− 98	− 44	+ 4					
…585	− 1 279	− 1 023	− 768	− 512	− 256	0	11.799	12.596	− 0.797	1 959	1 895
…947	− 777	− 613	− 457	− 299	− 145	+ 4					
…189.76	− 158.14	− 126.51	− 94.88	− 63.26	− 31.63	0					
…13.8	+ 9.8	+ 6.7	+ 4.6	+ 2.4	+ 0.6	− 0.5	2.221	1.756	− 0.465	911	1 138
…176	− 148	− 120	− 90	− 61	− 31	− 0.5					

Stress 22-22 $= \mp 0.05163\, m + v_s$
$= (-1)[\pm 0.05163\, m - v_s]$
$= (-1)[Z_0 + Z_1 + Z_2]$.

…VEN IN UNITS OF 1 000 LB.

+ Denotes tension.
− Denotes compression.

8	6	4	2	0	(31) Influence Area, $Z = \Sigma z$. Positive. +	(32) Negative. −	(33) Total.	(34) Live-Load Stress $= P\frac{y}{r}\Sigma z$. +	(35) −	
+ 410	+ 311	+ 213	+ 111	+ 4						
− 202	− 151	− 101	− 50	0	3.119	3.916	− 0.797	1 372	1 722	Above floor.
+ 208	+ 160	+ 112	+ 61	+ 4	3.119	4.783	− 1.664	1 372	2 105	Below floor.
− 218	− 164	− 109	− 54	0	2.691	3.488	− 0.797	1 325	1 718	Above floor.
+ 198	+ 147	+ 104	+ 57	+ 4	2.691	4.524	− 1.833	1 325	2 228	Below floor.
− 296	− 177	− 118	− 59	0	2.258	3.055	− 0.797	1 158	1 560	Above floor.
+ 174	+ 184	+ 95	+ 52	+ 4	2.258	4.054	− 1.796	1 158	2 070	Below floor.
− 360	− 195	− 180	− 65	0						
+ 150	+ 116	+ 88	+ 46	+ 4	1.928	2.725	− 0.797	894	1 264	
− 292	− 219	− 146	− 73	0						
+ 118	+ 98	+ 67	+ 33	+ 4	1.894	2.691	− 0.797	725	1 042	
− 327	− 246	− 164	− 82	0						
+ 83	+ 65	+ 49	+ 29	+ 4	1.818	2.615	− 0.797	663	954	
− 376	− 282	− 188	− 94	0						
+ 34	+ 29	+ 25	+ 17	+ 4	2.155	2.952	− 0.797	687	940	
− 450	− 336	− 225	− 113	0						
− 40	− 27	− 12	− 2	+ 4	3.344	4.141	− 0.797	855	1 050	
− 598	− 449	− 290	− 150	0						
− 198	− 138	− 86	− 89	+ 4	6.152	6.949	− 0.797	1 068	1 228	
− 1 122	− 848	− 551	− 281	0						
− 712	− 531	− 348	− 170	+ 4	16.330	17.127	− 0.797	1 295	1 348	
+ 2 080	+ 1 560	+ 1 040	+ 520	0						
+ 2 490	+ 1 671	+ 1 253	+ 631	+ 4	45.419	46.216	− 0.797	1 432	1 408	
− 65.0	− 49.0	− 33	− 16.0	0						
− 16.1	− 11.1	− 5.8	− 1.4	+ 1.2	4.196	6.083	− 1.887	879	612	
+ 269	+ 251	+ 174	+ 94	+ 5						

Stress (22-23) $= -0.2558\,(v_H - 0.17604\, m + 2.495\, v_o)$.
Stress (22-23)' $= -0.2558\,(v_H - 0.018305\, m + 2.495\, v_o)$
$= -0.2558\,(Z_0 + Z_1 + Z_2)$.

TABLE 17.—DETERMINATION OF DEAD-

Panel Length, $\lambda = 42.5$ ft. All Loads and S

PANEL POINTS.	(1) 0-1	(2) 2-3	(3) 4-5	(4) 6-7	(5) 8-9	(6) 10-11
Loads at top chord	+ 168	+ 187	+ 174	+ 157	+ 160	+ 155
Loads at floor height	+ 308	+ 261	+ 234			
Loads at bottom chord	+ 324	− 143.23	− 196.23	− 42.23	− 92.23	− 97.23
Total	+ 800	+ 304.77	+ 211.77	+ 114.77	+ 67.77	+ 57.77
Shear	+ 576.47	+ 273.70	+ 61.98	− 52.84	− 120.61	− 17
Moment = M	0	+ 576	+ 862	+ 914	+ 861	+ 741
TOP CHORD.	1-3	3-5	5-7	7-9	9-11	11-13
Stress, Case II, $-\frac{M\lambda}{r}$	− 228	− 408	− 593	− 558	− 529	− 441
BOTTOM CHORD.	0-2	2-4	4-6	6-8	8-10	10-12
Stress, Case II, $\frac{M\lambda}{r}$	0	+ 276	+ 477	+ 573	+ 567	+ 549
$\frac{L}{s_b} = \frac{\text{Bottom chord stress}}{\text{Length of member, in feet}}$	0	+ 5.12	+ 9.17	+ 11.41	+ 12.10	+ 11.69
$\frac{L'}{s_{b'}} - \frac{L}{s_b}$	+ 5.12	+ 4.05	+ 2.24	+ 0.69	− 0.41	− 1.80
DIAGONAL.	1-2	3-4	5-6	7-8	9-10	11-12
Stress $= s_d\left[\frac{L'}{s_{b'}} - \frac{L}{s_b}\right]$	+ 572	+ 365	+ 170	+ 47	− 26	− 109
VERTICAL.	0-1	2-3	4-5	6-7	8-9	10-11
$-s_v\left[\frac{L'}{s_{b'}} - \frac{L}{s_b}\right]$	Reaction for Case II = 1 400	− 716.80	− 457.87	− 208.17	− 55.20	+ 29.18
$C\frac{L'}{s_{b'}}$		+ 17.07	+ 30.56	+ 36.08	+ 40.29	+ 38.96
Lower panel point concentration	− 324	− 143.23	− 196.23	− 42.23	− 92.23	− 97.23
Stress in vertical below floor	− 1 076	− 848	− 653	− 212	− 107	− 29
Stress in vertical above floor	− 788	− 582	− 389			

PLATE XLIII.
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Dead-Load Stresses for Case II.
and Stresses Given in Units of 1 000 Lb.

(6)	(7)	(8)	(9)	(10)	(11)	(12)
10-11	12-13	14-15	16-17	18-19	20-21	22-23
+ 155 − 97.23	+ 135 − 148.23	+ 140 − 164.23	+ 140 − 165.23	+ 140 − 175.23	+ 137 − 182.23	+ 130 − 170.23
+ 57.77	− 8.23	− 24.23	− 25.23	− 35.23	− 45.23	− 40.23
.61	− 178.36	− 170.15	− 145.92	− 120.69	− 85.46	− 40.23
+ 741	+ 568	+ 392	+ 246	+ 126	+ 40	0
11-13	13-15	15-17	17-19	19-21	21-23	23-23A
− 441	− 336	− 228	− 124	− 41	0	0
10-12	12-14	14-16	16-18	18-20	20-22	22-22A
+ 549	+ 458	+ 341	+ 220	+ 125	+ 41	0
+ 11.69	+ 9.80	+ 7.65	+ 5.25	+ 2.90	+ 0.97	0
− 1.80	− 2.24	− 2.40	− 2.35	− 1.93	− 0.97	0
11-12	13-14	15-16	17-18	19-20	21-22	23-22
− 109	− 131	− 137	− 133	− 109	− 55	0
10-11	12-13	14-15	16-17	18-19	20-21	22-23
+ 29.16	+ 114.17	+ 127.23	+ 123.05	+ 110.10	+ 84.04	+ 40.09
+ 38.96	+ 32.96	+ 25.50	+ 17.50	+ 9.67	+ 3.23	0
− 97.23	− 148.23	− 164.23	− 165.23	− 175.23	− 182.23	− 170.23
− 29	+ 4	− 12	− 25	− 55	− 95	− 130

Length of $AB = s_b$
Length of $BC = s_{b'}$
Length of $DB = s_d$
Length of $AD = s_v$
Stress in $AB = L$
Stress in $BC = L'$
Ordinate C = [illegible] Ft.

To simplify the calculation, these ordinates were assumed as multiplied by the value, $\frac{r}{y}$; in other words, the ordinates, $A_1 A_2$ and $B_1 B_2$, were made equal to $\frac{x}{y}$ and $\frac{x'}{y}$, respectively (Fig. 55). The intermediate ordinates, Z_1, of the lines, $A_1 C_2$ and $C_2 B_1$, were then obtained by simple proportion from the ordinates, $A_1 A_2$ and $B_1 B_2$. (Tables 13, 14, 15, and 7.) These ordinates, Z_1, were then added algebraically to the influence ordinates, Z_0, for the horizontal reaction (polygon, $A_1 C_1 B_1$), in order to get the influence ordinates, Z, for the two-hinged arch condition.

To obtain any stress, the sum of the influence ordinates, Z, finally had to be multiplied by the coefficient, $\frac{y}{r}$.

3.—Live-Load Stresses.

The assumed live load is 6 000 lb. per lin. ft. of track, or 12 000 lb. per lin. ft. of truss, which gives a full panel load per truss of

$$12\,000 \times 42.5 \text{ ft.} = 510\,000 \text{ lb.}$$

To determine the maximum live-load stress in any member, the influence ordinates of the same sign were added (Columns 31 and 32, Tables 13, 14, 15, and 16), proper corrections being made for partial panel loads at the end vertical and at panel points adjacent to the zero points of the influence line.

The sum of the influence ordinates was then multiplied by the coefficient, $\frac{y}{r}$, multiplied by the panel load, 510 000 lb. (Columns 34 and 35, Tables 13, 14, 15, and 16).

4.—Dead-Load Stresses.

The arch was erected so as to act as three-hinged for all the dead load except the concrete and timber flooring, ballast, and tracks, which were placed after the trusses had been converted into two-hinged arches.

For the three-hinged arch, the stresses were determined as follows: First, the horizontal reaction was determined for the actual panel concentrations. Then, a uniform load was determined which would cause the same horizontal reaction. For this uniform load (16 000 lb. per lin. ft., or 680 000 lb. per panel per truss), the stresses were determined in the bottom chord members (Case I). As the bottom chord panel points are on a parabola, no other members are stressed for this case, and the stress in any bottom chord is equal to the horizontal reaction multiplied by the ratio between the length of the member and the panel length.

TABLE 18—DEAD-

All Stresses Given in Units of 1 000 lb.

BOTTOM CHORD.	0-2	2-4	4-6	6-8
Case I	— 11 457	— 11 023	— 10 616	— 10 239
Case II	0	+ 276	+ 477	+ 573
Total case, I + II	— 11 457	— 10 747	— 10 139	— 9 666
Case III	— 5 008	— 4 712	— 4 406	— 4 101
Total case, I + II + III	16 465	— 15 459	— 14 545	— 13 767
TOP CHORD.	1-3	3-5	5-7	7-9
Case I	0	0	0	0
Case II	— 223	— 408	— 523	— 558
Total case, I + II	— 223	— 408	— 523	— 558
Case III	— 84	— 199	— 340	— 488
Total case, I + II + III	— 307	— 607	— 863	— 1 046
DIAGONALS.	1-2	3-4	5-6	7-8
Case I	0	0	0	0
Case II	+ 572	+ 365	+ 170	+ 47
Total case, I + II	+ 572	+ 365	+ 170	+ 47
Case III	+ 216	+ 230	+ 224	+ 215
Total case, I + II + III	+ 788	+ 595	+ 394	+ 262
VERTICALS.	0-1 Below floor.	2-3 Below floor	4-5 Below floor	6-7
Case I	0	0	0	0
Case II	— 1 076	— 848	— 628	— 212
Total case, I + II	— 1 076	— 848	— 628	— 212
Case III	— 374	— 561	— 570	— 230
Total case, I + II + III	— 1 450	— 1 408	— 1 198	— 442
VERTICALS.	Above floor	Above floor	Above floor	
Case II	— 768	— 582	— 389	....
Total case, I + II	— 768	— 582	— 389	....
Case III	— 218	— 244	— 253	....
Total case, I + II + III	— 986	— 826	— 642	....

Horizontal Reaction for Case I = 8 672 900 lb. Bottom Chord Stress for Case I = Horizontal

Load Stresses.

8-10	**10-12**	**12-14**	**14-16**	**16-18**	**18-20**	**20-22**	**22-22′**
— 9 896 + 587	— 9 585 + 549	— 9 315 + 452	— 9 089 + 341	— 8 910 + 229	— 8 779 + 125	— 8 700 + 41	— 8 673 0
— 9 309	— 9 036	— 8 863	— 8 748	— 8 681	— 8 654	— 8 659	— 8 673
— 3 812	— 3 532	— 3 259	— 3 008	— 2 785	— 2 604	— 2 481	— 2 305
— 13 121	— 12 568	— 12 122	— 11 756	— 11 466	— 11 258	— 11 140	10 978
9-11	**11-13**	**13-15**	**15-17**	**17-19**	**19-21**	**21-23**	**23-23′**
0 — 529	0 — 441	0 — 336	0 — 228	0 — 124	0 — 41	0 0	0 0
— 529	— 441	— 336	— 228	— 124	— 41	0	0
— 636	— 793	— 951	— 1 103	— 1 234	— 1 332	— 1 384	— 1 287
— 1 165	— 1 234	— 1 287	— 1 331	— 1 358	— 1 373	— 1 384	— 1 287
9-10	**11-12**	**13-14**	**15-16**	**17-18**	**19-20**	**21-22**	**23-22′**
0 — 26	0 — 109	0 — 131	0 — 137	0 — 132	0 — 109	0 — 55	0 0
— 26	— 109	— 131	— 137	— 132	— 109	— 55	0
+ 222	+ 227	+ 225	+ 218	+ 185	+ 140	+ 77	— 138
+ 196	+ 118	+ 94	+ 76	+ 53	+ 31	+ 22	— 138
8-9	**10-11**	**12-13**	**14-15**	**16-17**	**18-19**	**20-21**	**22-23**
0 — 107	0 — 29	0 + 4	0 — 12	0 — 25	0 — 55	0 — 95	0 — 130
— 107	— 29	+ 4	— 12	— 25	— 55	— 95	— 130
— 197	— 181	— 158	— 127	— 87	— 39	+ 15	+ 166
— 304	— 210	— 154	— 139	— 112	— 94	— 80	+ 36
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....

$\text{Reaction} \times \dfrac{\text{Length of Member}}{\text{Panel Length}}$. + Denotes Tension. — Denotes Compression.

Then the stresses were determined for the difference between the actual and the uniform load (Case II). As, for both the actual and the uniform loads, the horizontal reactions are the same, that is, their difference is zero, it was possible to determine the stresses for Case II as for a simple span. The stresses in the chord members were determined from the bending moments. The stresses in the web members were then obtained by resolving stresses at the lower panel points. With the notations given in the diagram on Table 17 (Plate XLIII) we have, stress in diagonal $= s_a \left(\frac{L'}{s_b'} - \frac{L}{s_b}\right)$, stress in vertical below floor $= - s_v \left(\frac{L'}{s_b'} - \frac{L}{s_b}\right) + C\frac{L'}{s_b'} +$ panel load at bottom chord.

The stress in the vertical above the floor is found by deducting the panel load at the floor height from the stress below the floor.

For the two-hinged arch (Case III), the stresses were determined from the influence ordinates in a manner similar to that described for live loads. For panel concentrations, see Plate XXVIII.

Finally, the stresses for Cases I, II, and III were combined to obtain the total dead-load stresses (Table 18).

5.—Temperature Stresses.

Assuming the arch bearings as free to move longitudinally, a horizontal force of unity applied at each hinge causes a change in the span length equal to $\Sigma \Delta l$. A change in temperature of $t = 72°$ Fahr. causes a change in the span length equal to $\varepsilon\, t\, l = \frac{1}{150\ 000} \times 72 \times 977.5 \times 12 = 5.63$ in.

The horizontal reaction due to a change of temperature of 72° Fahr., therefore, is

$$H_t = \frac{\varepsilon\, t\, l}{\Sigma \Delta l} = 215\ 460 \text{ lb.}$$

The temperature stress in any member is then found as the product of H_t with the corresponding value, $S_1 = \frac{y}{r}$ (Table 19).

TABLE 19.—TEMPERATURE STRESSES FOR A VARIATION OF ± 72° FAHR.

Stresses Given in Units of 1 000 lb.

TOP CHORD.	1-3	3-5	5-7	7-9	9-11	11-13	13-15	15-17	17-19	19-21	21-23	23-23′
Temperature stress.......	± 72	± 170	± 291	± 416	± 543	± 677	± 811	± 940	± 1 052	± 1 136	± 1 180	± 1 157
BOTTOM CHORD.	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-22′
Temperature stress.......	∓ 285	∓ 363	∓ 463	∓ 573	∓ 683	∓ 802	∓ 925	∓ 1 049	∓ 1 166	∓ 1 269	∓ 1 349	∓ 1 416
DIAGONAL.	1-2	3-4	5-6	7-8	9-10	11-12	13-14	15-16	17-18	19-20	21-22	23-22
Temperature stress.......	∓ 184	∓ 196	∓ 192	∓ 183	∓ 190	∓ 194	∓ 192	∓ 181	∓ 158	∓ 119	∓ 66	± 29
VERTICAL.	0-1	2-3	4-5	6-7	8-9	10-11	12-13	14-15	16-17	18-19	20-21	22-23
Temperature stress.......	± 186	± 208	± 216	± 196	± 168	± 154	± 134	± 108	± 74	± 33	∓ 13	∓ 81

Horizontal reaction, H_t (for ± 72° Fahr.) = 215 460.
Stress in member = horizontal reaction × S_1.
For values of S_1, see Tables 7 and 8.

\+ Denotes tension.
— Denotes compression.

APPENDIX B

Financing, and Franchise of the New York Connecting Railroad Company.

A General Statement, Furnished by the Railroad Company.

The scope and cost of this extraordinary work and the great possibilities for transportation service which it opens up, justify a short record for public information as to the principal facts concerning the company under whose powers it was built, its incorporation, and its financing.

The railroad was originally conceived in 1892 by Oliver Barnes and Gustav Lindenthal, Members, Am. Soc. C. E., on practically the same location as that on which it was built.

The Certificate of Incorporation of The New York Connecting Railroad Company was filed and recorded in the office of the Secretary of State of the State of New York on April 21st, 1892, and provided for a steam railroad of about 10 miles in length, the termini of which were to be "in Westchester County, east of the Bronx River, and in the City of Brooklyn."

Among the incorporators were: Oliver W. Barnes, Frank M. Clute, Alfred P. Boller, Charles Macdonald, and Thomas S. King; the capital stock was placed at one thousand shares, par $100 000, all preferred stock.

In April, 1902, The Pennsylvania Railroad Company completed the purchase of the entire outstanding stock of the Connecting Company, and, in accordance with a prior understanding with The New York, New Haven and Hartford Railroad Company, a short time thereafter, sold one-half of this stock to that Company.

On June 11th, 1903, application for the franchise required from the City of New York was made by the Company to the Board of Rapid Transit Railroad Commissioners for that City, predecessor of the present Public Service Commission for the First District of the State of New York, and a tentative franchise was granted by that Board on June 23d, 1904, subject to the approval of the Board of Aldermen and the Mayor. After consideration by the Board of Aldermen for nearly a year, however, the proposed franchise was, on April 18th, 1905, returned to the Board of Rapid Transit Railroad Commissioners "disapproved."

On November 17th, 1905, the application to the Board of Rapid Transit Railroad Commissioners for a franchise was renewed, and, as a result of negotiations extending over a year, the franchise now held

by the Company, dated February 14th, 1907, was granted. This Certificate was accepted by the Company under date of February 28th of that year; was approved by the Board of Estimate and Apportionment of the City on March 8th (by amendments to the City Charter and Rapid Transit Act of the State, the duty and power to confirm franchises of this character had been transferred since the previous application from the Board of Aldermen to the Board of Estimate and Apportionment); was approved by the Mayor on March 14th, and, later, the Certificate was filed in the office of the Secretary of State of New York and in the offices of the Clerks of the Counties of New York, Queens, and Kings.

In addition to fixing the center line of the proposed railroad, the franchise, among other requirements, prescribed that the consent of the owners of one-half in value of the property bounded on the portions of streets crossed by the line, to the construction and operation thereof, should be obtained within one year from the acceptance of the franchise by the Company; that construction was to commence within 3 months after filing said consents with the Rapid Transit Board; and that construction was to be completed and the railroad in operation within 5 years after the commencement of construction. Provision was made in each case for extensions of time under certain conditions. The motive power prescribed to be used was steam, with the right to the Railroad Company to substitute electricity therefor, and, in addition, the Rapid Transit Board reserved the right, in the event of the use of steam constituting a nuisance or becoming dangerous to residents along the route, to require a change to electricity, or other motive power not less convenient to the public, within a period not to be less than 3 years after notice by the Board. The franchise fixes the rentals to be paid by the Company for the first period of 25 years from 2 years after obtaining the required consent of property owners (except for the use of Wards and Randalls Islands, in which case the rental was payable from the date of first occupation) and provides for a readjustment thereof at the end of said period and every 25 years thereafter. Said payments (as amended by order of the Public Service Commission in connection with the shortening of the franchise route of the Company from Knickerbocker Avenue to Fresh Pond Junction, hereinafter dealt with) are as follows: (*a*) For the right to construct and operate across streets and other public property other than Wards and Randalls Islands, \$23 925 per annum for the first 10 years of said 25-year period, and \$47 850 for the next 15 years; (*b*) \$100 per annum for the right to cross the East River between bulkhead lines; and (*c*) for the use of ground on Wards and Randalls Islands occupied, permanently and temporarily, during the

period of construction, by the abutments, piers, and other supports of the bridges and elevated structures, and for the use of portions of overhead space above the islands occupied by such bridges or elevated structures or for any other purpose, such annual payments as agreed upon by the Company and the Board of Commissioners of the Sinking Fund of the City, or other authorities in control of the islands. In addition to said rentals, the Company was compelled to file with the Comptroller of the City, within 60 days of the approval of the franchise by the Mayor, a bond in the sum of \$50 000, executed by it and by The Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company, as security for the performance by the Railroad Company of the terms and conditions of the franchise, especially with respect to the annual payments to the City, and had to pay a bonus of \$110 000 to the City within 60 days after the necessary consents of property owners had been obtained and before construction work was commenced.

On April 16th, 1907, the stockholders approved of an increase in the capital stock from \$100 000, Preferred, to \$3 000 000, Common, the holders of the Preferred stock having agreed to exchange their stock, par for par, for Common stock. The increase was approved by the Board of Railroad Commissioners on May 31st, and, as of June 1st, the exchange of Preferred stock was made and \$2 816 400 of the Common stock was issued, half and half, to the Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company, to take up a like amount of advances to the Connecting Company. Subsequently, the remainder of the authorized stock was issued, the entire \$3 000 000 thereof being held by The Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company in one-half proportions, all of the stock having been fully paid for at par.

The necessary consents of property holders to the construction of the line were filed with the Public Service Commission for the First District of the State of New York on July 20th, 1910, the said Commission having succeeded to the rights and powers of the Board of Rapid Transit Commissioners under the law. The original period fixed by the franchise in which to obtain said consents expired on February 28th, 1908, but, by certificate of the Commission dated February 3d, 1908, and March 5th, 1910, the time was extended to August 31st, 1910.

The period prescribed in the franchise for the completion of the railroad, including the Hell Gate Bridge, expired on October 20th, 1915, but the time was extended by the Commission to December 31st, 1917, and the same date was fixed by General and Special Laws of the State of New York.

The railroad as constructed conforms to the route prescribed in the franchise, except for a slight change of alignment between the north side of Wards Island and 132d Street, in the Borough of The Bronx, a change in grade between Broadway and Calamus Avenue, in the Borough of Queens, and a change in the southern terminus of the line from the Brooklyn Borough line to Fresh Pond Junction. The two changes first mentioned were approved by the Public Service Commission on July 19th, 1912, and maps covering the revisions were filed in New York and Kings Counties on July 23d, 1912, and in Queens County on July 24th, 1912; and the shortening of the franchise route to Fresh Pond Junction was approved by said Commission on June 7th, 1915; approved by the Board of Estimate and Apportionment on July 29th; by the Mayor on July 30th; and a map showing the modification was filed in the office of the Secretary of State of New York on August 13th, 1915, and also in the offices of the Clerks of the Counties of New York, Queens, Kings, and The Bronx. The approval of the War Department to the plans for the Hell Gate, Little Hell Gate, and Bronx Kill Bridges was granted by Certificate dated June 22d, 1906, and the changes in the plans for the two last mentioned bridges necessitated by the change in the route of the road across Randalls Island, hereinbefore mentioned, were approved by Certificate of April 4th, 1912.

In accordance with the requirements of the Franchise, the Company, on May 11th, 1907, submitted the plans for the Hell Gate Bridge to the Municipal Art Commission of the City of New York for its approval. By certificate of June 27th of that year, such approval was deferred, the Commission objecting to the proposed decorative treatment of the towers of the bridge. On May 29th, 1911, the plans were re-submitted to the Commission, the architectural features of the towers having been altered to meet the views of that body, and, by resolution of June 13th, 1911, the necessary approval was granted, a certified copy of which was, on August 7th, 1911, forwarded to the Public Service Commission for file, and receipt was acknowledged by it under date of August 8th.

The cost of constructing The New York Connecting Railroad, over and above the \$3 000 000 received from the sale of capital stock, was financed through the issue and sale of bonds covered by a first mortgage, dated May 31st, 1913, executed to the Guaranty Trust Company of New York, Trustee, for \$30 000 000, which was approved by the Public Service Commission by Order dated November 14th, 1913. Of these bonds, \$24 000 000 have been issued to this date and about \$1 500 000 additional will be issued; all are guaranteed, principal and interest, by The Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company.

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SAMUEL REA, W. W. ATTERBURY, GEORGE D. DIXON, W. H. MYERS, and A. J. COUNTY,	Representing The Pennsylvania Railroad Company.
HOWARD ELLIOTT, B. CAMPBELL, E. G. BUCKLAND, J. M. TOMLINSON, and A. R. WHALEY.	Representing The New York, New Haven and Hartford Railroad Company.

Mr. Samuel Rea (President of The Pennsylvania Railroad Company) is President of The New York Connecting Railroad Company; Mr. Howard Elliott (President of The New York, New Haven and Hartford Railroad Company) is Vice-President; and Mr. A. J. County (Vice-President of The Pennsylvania Railroad Company) is Assistant to the President of The New York Connecting Railroad.

DISCUSSION

W. H. Breithaupt,* M. Am. Soc. C. E. (by letter).—This magnificent structure, well described as of imposing magnitude, adds but another, if not the greatest, to the long list of achievements of its noted designer and chief engineer. The bridge, as likewise the whole New York Connecting Railroad of which it forms a part, will rank among the great things done in American engineering. Mr. Breithaupt.

Mr. Ammann's paper is a valuable addition to the literature of bridge engineering in its record and analysis of the bold design as a whole, and the clear-cut account of details and new departures, among which may be mentioned the 2-in. web plates and make-up of the main compression members, the large rivet sizes, the main arch bottom chord joint detail, the braking resistance, etc.

In the face of such excellence of design and execution, it appears graceless to criticize; however, a few remarks on the general design may be in place.

The dignified and beautiful abutment towers of the main arch are wholly admirable, more especially when compared with the earlier design, Fig. 8, with its suspended sign-board keystone, its meaningless surface elaborations, its flaring base, its lack of form as a whole.

For the Little Hell Gate Crossing, deck-truss, parallel-chord spans, as at first designed, but ruled out by the War Department on account of navigation requirements, would have made the most fitting structure. Arches are not discussed. It is not clear why a two-hinged, spandrel-braced arch design, or two-hinged, braced, double-rib arches, should not have been used. A low-spring, high versed-sine arch design would have effected a considerable saving in masonry with little or no addition in superstructure weight. Aside from the floor support struts, high at the ends, the superstructure frame could have been of less weight. Erection could have been balanced outward from the piers—tied from end abutments—the material floated to position underneath and hoisted, with only a few bents under end-panel points each way, thus effecting a large saving in expensive falsework. The skew interference could have been taken care of as well as, or better than, with the bowstring trusses. The bracing could have been better. The bowstring spans have no lateral bracing along the bottom chords; the stiff bracing between the trusses, although without bottom transverse struts, is, in the description of the main arch bracing, rightly deprecated as tending to distortion under one-sided loading. Expansion would have been taken care of separately for each span. The relative moveme of ± 6 in. at the center pier of the bowstring spans, as describe unduly large. With arches, the high clearance for navigatior have been at the center of the span, with plenty of room f

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Mr. Breithaupt. movement of tow, instead of danger of crowding against the piers, as with the present spans. On the other hand, the present spans, for the greater part of their length, give fair clearance for ordinary navigation. For navigation, in fact, as well as for general appearance, three arches for the crossing would have been the preferable design. Lastly, there is the reason, the more deserving of consideration in so monumental and visually prominent a structure, of harmony of general design. The inverted arch effect of the bowstring girders, so near the main arch, involves something in the nature of mental gymnastics.

The gate-post design of the towers marking the ends of the Little Hell Gate Crossing is not entirely satisfactory, either in photograph or to the eye *en plein*. A conceivable purpose would have been as shelters and material or implement stores for watchmen, sentries, or workmen, and a design, lower while retaining massiveness, would have been in keeping.

Mr. Moisseiff. LEON S. MOISSEIFF,* M. AM. SOC. C. E.—This paper is of unusual importance in the literature of bridge building, and deserves attention and study. It occurred, under a fortuitous constellation, that a first-class railroad, provided with ample financial resources, undertook to build a short connecting line in the metropolis of the Western hemisphere. It, of course, decided to build the new line in a first-class manner; and, as part of the line happened to require the erection of a long-span bridge capable of carrying a very heavy traffic, the management of the road, in its wisdom, selected a first-class bridge engineer with a broad vision and decided scientific leanings. Adding the opportunity of ample time to conceive, to plan, to compare, to design, and to re-design, and again adding to the plans and specifications the resources in mind and matter of one of the greatest bridge manufacturing concerns of the world, the resulting product is such that American engineers may be proud of the achievement. The longest arch bridge in the world, having a span of nearly a thousand feet, has been planned and built so as to produce an imposing and pleasing structure, well proportioned in outlines as well as in details.

The paper is full in its description and, as it takes up successively the consideration and treatment given to the design in general and to its details, numerous points are brought out worthy of commendation and discussion. Many of the vital points of bridge engineering are touched, too many indeed to be given the discussion they deserve.

The comparative designs of the several types of bridges, all with a central span of 850 ft., are most instructive. The stiffened suspension type with eye-bar chains has shown off remarkably well, for a span comparatively small for a suspension bridge, such bridges being best adapted for the longest spans. The good showing made by this type is

* New York City.

due to the eye-bar design. If built, it would have resulted in a fine and economic bridge. Somehow, from reading the paper, one obtains the impression that if the difficulties encountered in one of the abutment foundations had been known at the time of selecting the type, the suspension bridge with a longer span, and not the arch, would have been chosen. Mr. Moisseiff.

The difficulties encountered in the foundation of the Wards Island abutment were unusual, and, if known, the foundations would certainly have been avoided. The appearance of the masonry towers flanking the steel arch is good, but their use, to restrict the size of the foundation and by their weight to obtain a steep resultant to pass within the middle third of the base area, can hardly be claimed to be economical.

Very properly the bridge was erected as a three-hinged arch for most of the dead load and was made two-hinged for moving load and temperature only. In connection with this the author remarks: "This, however, is a convenience in erection rather than advantage as regards stress action." This statement is true where the abutments are absolutely unyielding, but, where any limited settlement may be apprehended, erecting the arch first as three-hinged has a more substantial advantage than mere convenience of erection. It has the purpose of eliminating stresses caused by yielding, which stresses may become uncomfortably large. In that case it is good policy to leave the arch three-hinged for at least a year, so that all probable settlement and yielding may have taken place and be eliminated, and, after that time, when no further yielding is observed, to make the arch two-hinged in order to obtain the advantage of increased stiffness.

Making the bottom chord a massive rib sustaining much of the dead load is excellent designing and testifies to the engineer's clear conception of the stiffening function of the truss members of a braced abutment without straining the remaining truss members. This is good mechanics as well as economic design.

The floor lateral truss and the braking girders are additional instances of well-considered design.

The use of 2-in. plates in compression members is extraordinary. The claim made that metal of such thickness showed in tests the same elasticity and ultimate resistance as ½-in. material is contrary to general experience. If the manufacturers were able to roll, of the same material, 2-in. plates of strength equal to the usual structural sizes, it would prove of great interest to the engineering world. Hitherto it has been held that the heavier plate receives less work and shows considerable decrease in strength. Should this claim be substantiated, the use of stitch rivets would decrease considerably, and heavy plates would come into use extensively for columns. This is a matter of much practical importance, and deserves a full discussion by the steel makers.

Mr. Moisseiff.

The "Rules of Design" laid down by Mr. Lindenthal are so interesting that the speaker regrets that they have not been appended in full to the paper.

The three-face joint of the bottom chord is a novel feature in compression chord bearing, and is a move in the right direction to reduce secondary stresses caused by erection deflections. High edge stresses are thereby avoided. The speaker, however, is of the opinion that the concentration of stress on the middle portion should be taken care of in the allowable unit stress. In other words, the computed excess stress should be subtracted from the specified unit stress for the compression members.

The principle adopted by the designer for loads and unit stresses is the right one for long-span bridges. To take into consideration all possible forces and causes, including even an allowance for secondary stresses, and then to fix a proportionately high unit stress is a step in the direction of scientific approximation of actual conditions in the bridge. Of course, it will require much study and consideration to arrive at a final judgment as to the proper unit stress, but, considering all factors, a nearer approach to true conditions will be attained. At the initiative of Mr. Lindenthal, this principle has been partly applied to the Queensboro' Bridge and entirely to the Manhattan Bridge, and now has been adhered to in the Hell Gate Arch. He deserves credit here for his clearness of vision.

A step out of the ordinary is the proportioning of compression members on the basis of net area, deducting the rivet holes. Good reasons for the unusual procedure are brought forward in the paper. The matter, however, is of such extraordinary importance, affecting the design and the economy of all steel structures, that it requires careful research and exhaustive testing to adjust and co-ordinate present standards and specifications. The speaker hopes that the Society's Special Committee on Steel Columns and Struts will give the matter the consideration it deserves.

The speaker has attempted to touch only a few of the most important points in the paper. Many other interesting innovations which it contains are deserving of full discussion.

The paper is well presented, and full of information, and Mr. Ammann deserves the thanks of the Profession for his work.

The Hell Gate Arch Bridge is an excellent example of what engineering genius can accomplish if the project is entrusted to one mind to plan and direct, unhampered by red tape and lay commissioners. The bridge reflects credit on the Engineering Profession, and much praise is due to Mr. Lindenthal and his able associates and co-workers, as well as to the engineers of the American Bridge Company. They have done well.

SAMUEL T. WAGNER,* M. AM. SOC. C. E. (by letter).—The thanks of the Profession are due to Mr. Ammann for the admirable manner in which he has presented the data of the conception, design, and execution of this most monumental structure. It is possible to find answers to nearly all the questions which arise in its careful reading. This is quite unusual in a work of this magnitude and character. Mr. Wagner.

The use of plates 2 in. thick in built-up members is so out of ordinary practice as to attract special attention. It is hoped that, in closing, Mr. Ammann will see fit to give some of the actual tests of this material. The writer's experience has been that such thicknesses are likely to produce material which is not wholly reliable. Years ago, plates 1 in. in thickness of satisfactory quality were difficult to obtain, especially before the use of open-hearth steel was general. If Mr. Ammann could give the details of the manufacture of these plates, it would be specially interesting. If it is possible to obtain such sections of good quality, even at slightly increased cost, it will open the way for many details which have not been considered possible up to the present time.

The details of the latticing are interesting. There are many advantages in the use of stiff latticing on members of any considerable size. Many members are designed with such light and inadequate lattice bars that even the most careful handling in the shop and during erection results in damage to them; and, frequently, splendid work in the shop is spoiled before it is placed in the structure. The spacing and general arrangement of the latticing are also to be commended.

The facing of the ends of such large members is a work that requires unusual care in the shop. There can be nothing more important than proper bearing in compression members, and the machine work required, in the writer's opinion, is one of the most difficult parts of the shop fabrication that has to be done on such a structure. It would appear, from the stress measurements made on the completed structure and given in the paper† by Mr. Steinman, that this part of the fabrication is specially commendable on the part of the manufacturers. The details of the joints in the bottom chord are most unusual and interesting, and indicate the great care which was taken in planning them. The results undoubtedly confirm the wisdom of the design.

The method of drilling the main members by punching a limited number of holes, using tack-bolts, and drilling the majority of the holes through the solid, is good practice. There is no doubt in the writer's mind, judging from his past experience, that this is the proper way to do work of this character, not only from the standpoint of doing good

* Philadelphia, Pa.

† *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 1040.

Mr. Wagner. work, but also of doing it in the most economical and workmanlike manner. It is specially true for the higher carbon steel which was used.

The assembling of the arch in sections is also another most excellent detail of the fabrication. It would be interesting to know whether this was a requirement of the specifications. For arch work, it is doubtful whether such details could have been fabricated satisfactorily in any other way. The writer's first experience on work of this character was on a very small scale in connection with the three-hinged arch trusses of the train-shed of the Reading Terminal in Philadelphia.* In this case, the complete half arches were assembled in the shop. If this had not been done, there surely would have been bad workmanship and difficulty in the erection. It is a great comfort to any manufacturer to know that the joints thus assembled are sure to match. This is especially true in the case of the Hell Gate Arch, with its difficulties in erection.

The use of ballasted floor construction is to be commended, although its cost is greater than the other types referred to in the paper as having been considered. The reasons given for its adoption are well expressed, namely, "more uniform roadbed, less noise and impact, smaller cost of maintenance, and greater safety in case of derailment or fire." Although it seems to be difficult to show in actual figures the desirability of such construction from a financial standpoint, there can be no doubt that under the conditions in this case the additional cost was fully warranted. The advantage of having a standard track in place of special ties is important. Experience in viaduct construction in which a solid floor was used has shown conclusively that it is the best known detail for the prevention of noise. In a number of lawsuits, in which damage from noise has been claimed, no attempt has ever been made to produce proof. It is rather unusual at this time to see a floor of this type designed so that there is sufficient steel to carry the loads without any allowance for the use of the embedding concrete. It is a detail, however, of which the writer approves, especially in such a thin floor. He would have gone one step further, however, in that the floor would have been water-proofed in order to give additional durability to the construction, and it is believed that the additional cost would have been fully justified. If, by some means, water gets through the concrete and to the beams, a condition is created which is very unpleasant to contemplate. After traffic has once been placed on a structure, the difficulties of making any repairs to the floor are very serious.

The officials of the New York Connecting Railroad are to be congratulated on having the foresight and financial courage to design and construct this bridge for four tracks instead of two. It is a monument to all concerned.

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* *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 181.

CHARLES EVAN FOWLER,* M. AM. SOC. C. E.—The speaker's acquaintance with Mr. Lindenthal and his work began about 30 years ago, and the genius displayed in his designs for the Seventh Avenue and Smithfield Street Bridges, in Pittsburgh, led one to expect the design by him of some great structure like the Hell Gate Arch. The speaker hopes that Mr. Lindenthal may live to see an even greater culmination of his career, by the building of the great suspension span proposed for the North River. Mr. Fowler.

The Seventh Street Bridge, in Pittsburgh, built in 1884, is a two-span, eye-bar, suspension bridge, with braced parallel chains, and was a milestone in the design of suspension bridges. The two main spans are 330 ft. each, and the side spans 165 ft. each, with a width of 42 ft.

The Smithfield Street Bridge, which is still in use, is a very striking structure, consisting of two Pauli truss spans, the longest and heaviest of this type in America.

That the younger members of the Profession may not forget to have pride in American bridge engineers, it is well to call to mind the work of the Dean of this branch of the Profession, the late C. Shaler Smith, M. Am. Soc. C. E. His Kentucky River High Bridge was a wonderful structure, with three 375-ft. spans, 275 ft. above the river, a continuous-span bridge, which, by the cutting of the chords to fix the points of contraflexure, became one of the first great cantilever bridges. The four river spans of his Lachine Bridge, over the St. Lawrence River, were at the same time an example of the most scientific bridge engineering and an altogether striking and beautiful structure. The main bridge had two 408-ft spans, with two 269-ft. side spans.

When this structure required rebuilding, within the last few years, in order to carry modern heavy loading, it was with regret that we saw that its outlines were not preserved. When Mr. Lindenthal was called on to replace High Bridge with a heavier structure, a few years ago, the speaker, for one, felt that a vote of thanks was due to him for preserving the original outline. His work on the bridges of New York City, although, unfortunately, hampered in many ways, is reflected in the completed Hell Gate Bridge, which is the first wholly creditable structure to be built here since the completion of the old Roebling Bridge.

To the speaker the outline of the structure is most pleasing. This is due not only to the smooth curve of the arch proper, but mostly to the reverse curve of the top chords near the towers. The combination resembles, and is as pleasing as, a similar combination of curves in the "Camelback Bridge" in the Imperial Palace grounds at Pekin, China. To the speaker, the latter is the most pleasing of Oriental bridges.

* New York City.

Mr. Fowler.

The sickle-shaped truss design, which was abandoned for the Hell Gate Arch, for reasons having to do mainly with erection, could hardly have been more pleasing in its architectural appearance.

The towers as built are plain and simple in design, but are in every way appropriate and in harmony with the great arch. The speaker's reason for calling attention to this is the hope that some artistic person in the New York City government or the Art Commission may some day succeed in getting the parapet, or "attic story" as it might be termed architecturally, added to the towers of the Roebling Bridge, as originally planned.

Many great bridges have been marred architecturally by the appearance of the approaches, but when one visits this structure at Hell Gate and views the great masonry approach piers in perspective, their pleasing and appropriate character is fully appreciated. The whole structure may be said to be the most complete lesson in bridge esthetics in America, and one of the best in the world. The proper consideration of simplicity, symmetry, harmony, and proportion in bridge design was first enunciated by the speaker in "Engineering Studies", and, as it has been elaborated in many subsequent writings and discussions, he will take time only to repeat that the failure to regard any one of these four fundamentals must lead in some degree to failure.

The Dusseldorf and Bonn Bridges are both notable examples, which have been referred to by Mr. Ammann, but the Bonn design, especially when the bridge is seen in perspective, comes nearest to fulfilling all requirements, including one other, that of an unequal number of spans, the main span of 614 ft. and the side spans of 307 ft. There are so many notable arch bridges in France, both of masonry and steel, that it would be presumptuous in a limited discussion to do more than call to the attention of designers this great field of study. The great arch span at Oporto, with both upper and lower roadway, is perhaps the most striking of any of the examples mentioned by Mr. Ammann.

The great 840-ft. Clifton Arch at Niagara is one of the best examples in America, but it is with regret that one views the very inappropriate approach spans of the wonderful Grand Trunk Arch, its near-by neighbor. The Eads Bridge, across the Mississippi at St. Louis, with its three great steel arch spans of about 500 ft. each, with simple, appropriate, and well-designed masonry approaches, answers all the basic requirements of an artistic design.

There are five designs of the Quebec crossing which should be preserved in the final monograph of that great bridge. First, the cantilever design of many years ago, by Brunles and Light, a 1 442-ft. span for which much of the credit was due to Professor Fidler. Second, the 1 800-ft. cantilever which failed, due to neglect to observe fundamental features of design for compression members, which had been discussed previously by the speaker's one-time assistant, the late Pro-

Mr. Fowler.

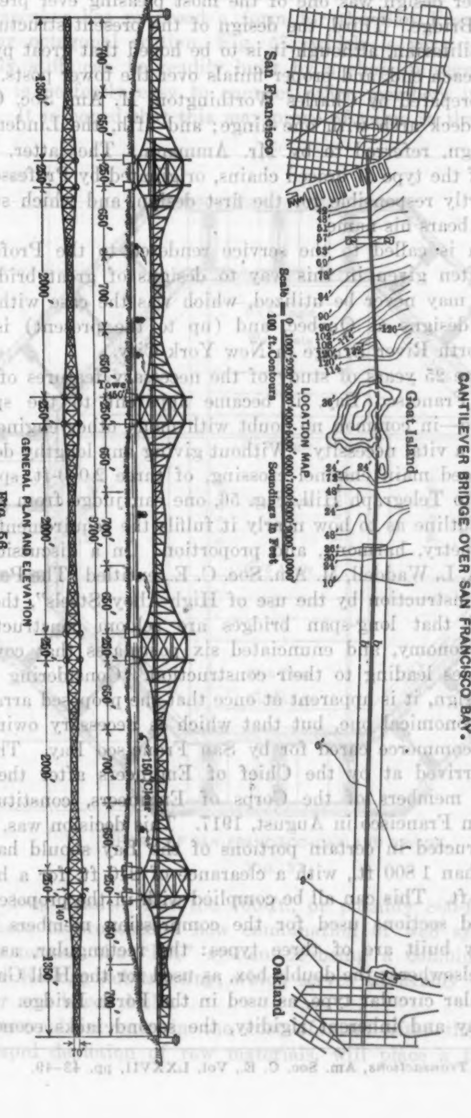


FIG. 56.

Mr. Fowler.

fessor A. H. Heller, in his "Stresses in Structures". The outline of this cantilever design was one of the most pleasing ever prepared for the Quebec Bridge. Third, the design of the present structure, which is strictly utilitarian, although it is to be hoped that great pylons will be added at each end, and proper finials over the tower posts. Fourth, the design prepared by Charles Worthington, M. Am. Soc. C. E., for an 1 800-ft. deck arch with one hinge; and fifth, the Lindenthal suspension design, referred to by Mr. Ammann. The latter, curiously enough, is of the type of braced chains, originated by Professor Fidler, who was partly responsible for the first design, and which suspension type usually bears his name.

Attention is called to the service rendered to the Profession by the study often given in this way to designs of great bridges, even though they may never be utilized, which was the case with three of the notable designs at Quebec, and (up to the present) is true as regards a North River Bridge in New York City.

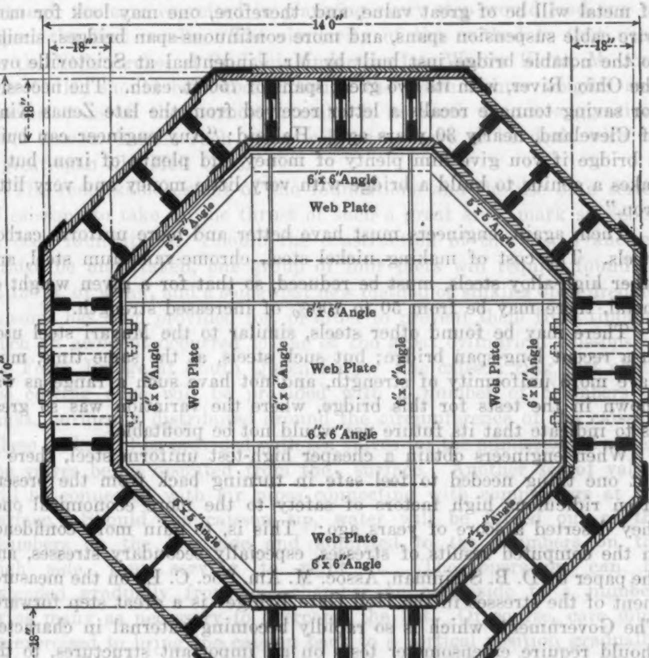
After some 25 years of study of the necessary features of a bridge across San Francisco Bay, it became apparent to the speaker, a few years ago—in common no doubt with many other engineers—that a bridge was a vital necessity. Without giving any lengthy description of the proposed main channel crossing, of three 2 000-ft. spans from Goat Island to Telegraph Hill, Fig. 56, one can judge from an inspection of the outline as to how nearly it fulfils the requirements of simplicity, symmetry, harmony, and proportion. In a discussion of the paper by J. A. L. Waddell, M. Am. Soc. C. E., entitled "The Possibilities in Bridge Construction by the use of High-Alloy Steels", the speaker pointed out* that long-span bridges are seldom constructed from motives of economy, and enunciated six postulates that covered the basic principles leading to their construction. Considering this San Francisco design, it is apparent at once that the proposed arrangement is not the economical one, but that which is necessary owing to the great ocean commerce cared for by San Francisco Bay. This is the conclusion arrived at by the Chief of Engineers after the hearing before three members of the Corps of Engineers, constituting the Board, at San Francisco in August, 1917. This decision was, that any bridge constructed in certain portions of the Bay should have spans of not less than 1 800 ft., with a clearance of 210 ft. for a horizontal width of 800 ft. This can all be complied with in the proposed design.

The chord sections used for the compression members of great spans already built are of three types: the rectangular, as used at Quebec and elsewhere; the double box, as used for the Hell Gate Arch; and the cellular circular type, as used in the Forth Bridge. The first lacks economy and inherent rigidity, the second lacks economy, but

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, pp. 43–49.

the third seems to lack nothing, theoretically, but practically is a very expensive form to build and very difficult to join with other members. Mr. Fowler.

The speaker has proposed a form for the San Francisco Bridge, a cellular octagonal section, Fig. 57, which is very economical, is inherently stiff, can be readily built by a shop in segments, and to which it is perfectly easy to connect other members in an efficient manner. It is hoped that this may prove of value in the construction



CHORD OF SAN FRANCISCO-OAKLAND BRIDGE.

FIG. 57.

of many long spans. For the 700-ft., or possibly 800-ft., suspended spans, there is proposed a new combination of web systems, which has been termed the "PK" system, it being a combination of the Petit and Kellogg sub-trussing, which serves admirably to carry both the upper and lower decks.

The normal increase in the cost of steel after the war, owing to the rapid depletion of raw materials, will place a time limit on

Mr. Fowler. the period within which great spans may be constructed, and, in a conversation with Mr. Lindenthal recently, he pointed out that this period would not be likely to extend beyond 30 years, or, say, 1950. Therefore it behooves engineers, not only to be more conservative in the use of steel, but also to be less prodigal; they should plan in every way to extend the time of depletion.

The use of such types of structures as will require the least weight of metal will be of great value, and, therefore, one may look for more wire cable suspension spans, and more continuous-span bridges, similar to the notable bridge just built by Mr. Lindenthal at Sciotoville over the Ohio River, with its two great spans of 750 ft. each. The necessity for saving tonnage recalls a letter received from the late Zenas King, of Cleveland, nearly 30 years ago. He said: "Any engineer can build a bridge if you give him plenty of money and plenty of iron, but it takes a genius to build a bridge with very little money and very little iron."

Then, again, engineers must have better and more uniform carbon steels. The cost of making nickel steel, chrome-vanadium steel, and other high-alloy steels, must be reduced, so that for a given weight of metal, there may be from 50 to 100% of increased strength.

There may be found other steels, similar to the Mayari steel used in a recent long-span bridge; but such steels, at the same time, must have more uniformity of strength, and not have such a range as was shown in the tests for this bridge, where the variation was so great as to indicate that its future use would not be profitable.

When engineers obtain a cheaper high-test uniform steel, there is but one thing needed to feel safe in turning back from the present often ridiculous high factors of safety to the more economical ones they deserted a score of years ago: This is, to gain more confidence in the computed results of stresses, especially secondary stresses, and the paper by D. B. Steinman, Assoc. M. Am. Soc. C. E., on the measurement of the stresses in the Hell Gate Bridge* is a great step forward. The Government, which is so rapidly becoming paternal in character, should require extensometer tests on all important structures, to the end that engineers may eventually feel perfectly safe in designing all steel structures with very much greater unit stresses and a consequent enormous saving in steel consumption.

The specifications as to shop work are worthy of special study in connection with all designs for future structures of any importance whatever, and the clauses with reference to punching, reaming, and drilling are especially noteworthy to one who has spent years in the fabrication of steelwork. The data in regard to the driving of long rivets should be read with care, and great good would result from the application of these methods to all railway and highway bridges of

* *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 1040.

importance. Unless the engineer has had occasion to rebuild some old bridges, he cannot realize how criminally careless the average bridge erector is in driving all rivets with a long grip. Mr. Fowler.

This subject is of such great interest that it is impossible to close without congratulating all those having to do with the erection of the Hell Gate Arch on the unusual success of all the operations, although the care exercised in planning every step would lead one to expect the best results. The similarity of the method to that used for the Eads Arches at St Louis under the late Walter Katte, M. Am. Soc. C. E., should be noted, although the much lighter weights and shorter spans of that structure make it no more than a suggestion, as to how to handle such great weights as have to be dealt with in a great span like Hell Gate.

The remarkable extent of the foundations and the use of a series of caissons to take up the thrust of such a great arch, mark an epoch in deep foundations. Should the construction of the San Francisco Bridge be undertaken, one group of four piers will require founding in 130 ft. of water, and a somewhat new method of sinking open dredged caissons has been provided for, in order to prevent them from tipping. Such an accident occurred to one of the great cylindrical caissons of the Forth Bridge at South Queensferry. The caisson ring just above the cutting edge will be provided with a number of chambers or tanks, uniformly distributed around the circumference of the caisson. These tanks will be provided with sea valves for the admission of water, the valves being operated from the surface. Another set of valves will be connected with air pipes connecting with compressors at the surface. Should the caisson tip, water will be forced out of the chambers on the low side and admitted to the chambers on the high side, thus serving to right it. The operation can be extended gradually from one chamber on each side to a number, or as many as necessary to overcome the list. Of course, care must be exercised to bring the caisson back to a vertical position gradually, and to meet the movement by a reversal of conditions in the opposing chambers. The term "ballmatic" has been applied to this type of pier, the name being coined by a combination of the words "ballast" and "pneumatic."

The speaker realizes that papers having such a wide scope as these, describing this great bridge, and the stress measurements of the members, will call forth a very extended discussion, and that lack of space will require their curtailment. Thus, many features that could be profitably considered will have to be omitted, and the speaker will close in the hope that others will cover many other points of value, and of vital importance for consideration, in the design of future long spans.

Mr. Quimby. HENRY H. QUIMBY,* M. AM. SOC. C. E.—In the presence of the magnificent achievement of this imposing monument, the product of the very highest grade of technical and practical skill in the world, through years of study under apparently ideal conditions, with *carte blanche* as to time and cost, and free play for fancy and cultivated taste, one hesitates to introduce a note of comment that may even seem like criticism. In the published writings on the work, however, in the paper as presented, and in Mr. Lindenthal's supplemental oral remarks, much emphasis was laid on the matter of appearance, and it seems that it was not so much economy as esthetic considerations which controlled in the selection of the type of structure in at least three of the divisions of the work—the main span, the viaduct approaches, and the Little Hell Gate Bridge. Wherefore, it is probable that the designers, masterly and successful as they are, will welcome a frank expression of judgment on their taste.

Esthetic considerations embrace the effect on the lay mind as well as on that of professionals; because of this, public Art Commissions are created to pass on technical construction—on the work of artists as well as that of mechanics—of architects as well as of engineers. It is desirable, therefore, to make a structure appear not only graceful but satisfyingly stable to the miscellaneous eye as well as to the trained and understanding scientific eye. One feature of the main span suggests these reflections. The end post of the spandrel bracing in bearing on the end pin is almost directly over the front face of the abutment support, and to the miscellaneous mind this post appears to be the end of the bridge—the span structure. The effect, when viewed from a point at right angles to the bridge, is to make it seem as if the truss is resting just on the edge of the bridge seat, and with the considerable space between it and the shaft of the abutment pier, the truss looks short for the span opening.

The earlier published plans showed the top chord terminating at the end post, leaving a gap between it and the shaft, which condition accentuated the impression of scantiness of length of truss. The subsequent extension of that chord into the shaft relieved this feeling and much improved the appearance. As a false member was thus inserted at the top, the improvement might have been continued by adding a panel, or a false diagonal, from the end pin to the shaft, but as the end post has no necessary relation to the end pin in its present location—which evidently was fixed to secure adequate distribution of bearing on the skewback—it would seem to have been better to reduce the gap by moving the post 3 or 4 ft. closer to the shaft, so that its direction would intersect with the direction of the thrust of the end lower chord member at a point nearer the skewback, and then give it a separate pin. This would not complicate the determination

* Philadelphia, Pa.

of stresses, and it would give the structure the appearance to everybody of adequate length and bearing on the bridge seat. The length of each panel and the theoretical length of the span would thereby be increased very slightly, but the angle of thrust is so steep that the stresses in the arch would probably not be increased sufficiently to constitute an objection to the change. Mr. Quimby.

A view of the approach piers during their construction, and before any of the steelwork was on them, gave some the impression of disproportionate massiveness compared with the spaces between them, and this feeling still continues with the superstructure in place. Would not such height and volume of masonry, in the case of the higher piers, more economically and more pleasingly support longer and heavier spans? We are not told whether the pier spacing was determined by economy of proportion between superstructure and substructure, or by right-of-way conditions. The exhibit of alternative designs with steel bents in itself justifies the type adopted and used, for the rocker bents—absolutely without a base in the side elevation—would give a very unpleasant feeling of instability. The ordinary public observer would recognize at once that somewhere something must be provided to make up for the palpable deficiency in longitudinal stability of the individual bents, and the Art Commission's objection to the steel designs shown is, of course, clear to all; but, was a trial made of a design of tapering steel towers—steel bents with buttresses—each stable in itself against longitudinal as well as transverse forces, and, if so, how did it compare in cost, including the capitalized cost of maintenance? The paper gives the reasons for not adopting concrete arches for the approaches, and the soundness of such reasons will probably not be questioned, but the appropriateness of that type of construction there has probably occurred to almost every interested observer.

One other feature of the work is of character and proportions that do not accord with the taste of some students of bridge architecture. This is the bridge over Little Hell Gate. The inverted bowstring trusses introduce a note that, although not really discordant, seems very outstanding, and, in dipping down so close to the water, they give the impression of superstructure depth disproportionate to the whole-structure height. The depth of construction at mid-span is quite appreciably more than half of the whole height of the structure above the water. The paper states that the trusses were made of the bowstring type so as to give additional clear height near the piers for vessels to pass—a somewhat unusual provision for navigation which generally prefers to give channel piers a wide berth. As the foundation here was on rock at a reasonable depth, it would seem that arches of steel, or even of concrete, might have been adopted as consistent and pleasing, and less obstructive to navigation. Was an arch design investigated, and, if so, how did it compare with the bowstring?

Mr. Quimby. The design of the Bronx Kill Bridge, with inclined end posts at the abutments and vertical ones at the pier, suggests the case of more than one bridge where an original single span has been converted into two spans by building a pier under its middle after it had become inadequate. Presumably, the heels or counterweight trusses (the design of which is not found in the paper) will, when placed, remove the present disparity, but, as that may not be for many years, would it not have been worth while to make the trusses symmetrical in outline during the interim by inclining the pier ends, which is a common practice for the outer ends of bascule leaves.

The paper is an unusually satisfying one, both in the fact that it appears while the public and the professional interest in the remarkable feat is still fresh, and in that it discusses so freely the reasons for the various features of the design. The oral presentation of the subject by the author was also exceptionally felicitous, summarizing and supplementing the paper rather than repeating it by reading word for word, as is too often done with preprinted papers.

Mr. Seaman. HENRY B. SEAMAN,* M. AM. SOC. C. E. (by letter).—This description of the largest, the most scientific, and, it is believed, the most artistic bridge yet constructed, has been so complete that anything added might seem to be a presumption. It is probably the first large construction wherein was adopted the new method of proportioning, by which a static unit strain is allowed and the excess due to the action of the live load is provided for entirely by an impact formula. This method of proportioning is the most rational and the most scientific which has yet been used, and receives its ultimate interpretation in the present structure. After investigating the subject very thoroughly, while acting as Consulting Engineer to the Department of Bridges, New York City, the writer, later, took occasion to present the result of such study to the Society in 1912.† It seems to be but a progressive step from that which was first instituted some thirty years ago by the late Charles C. Schneider, Past-President, Am. Soc. C. E.

The description of the higher scientific characteristics of this structure have been thoroughly outlined by Mr. Ammann, but it might not be amiss, in a short space, to mention the methods adopted in erecting the tall masonry piers of the approach viaduct.

The foundations of these piers were not unusual in any way, but, being of reinforced concrete, of an extreme height of about 125 ft., and in a continued series about 90 ft. apart, the method of constructing them rapidly necessitated some new thought, or, as might be better expressed, new application of old principles.

* Washington, D. C.

† "Specifications for the Design of Bridges and Subways", *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 313.

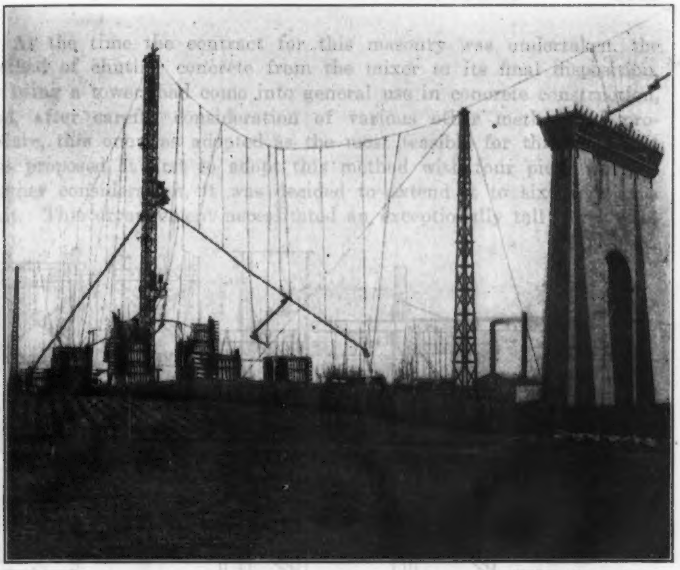


FIG. 58.—GENERAL ARRANGEMENT FOR CHUTING CONCRETE.

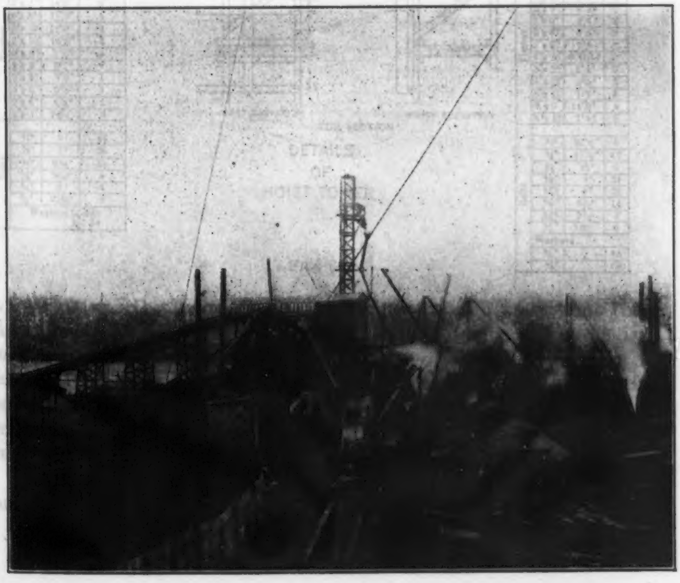


FIG. 59.—WARDS ISLAND UNLOADING PLANT.

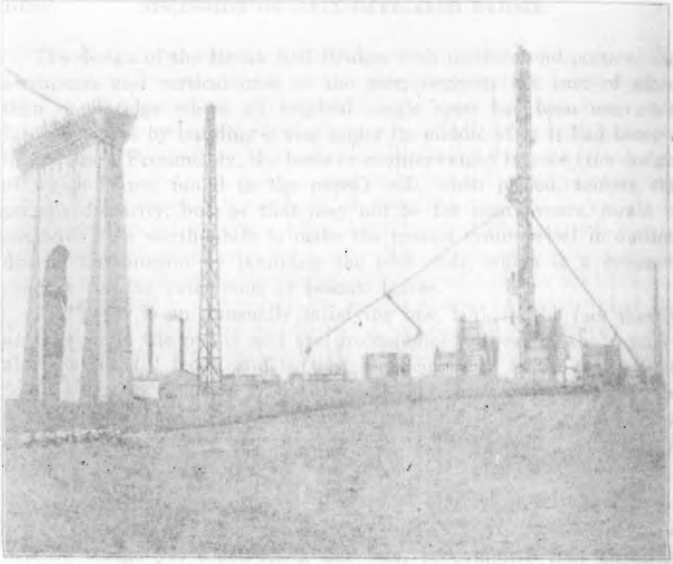
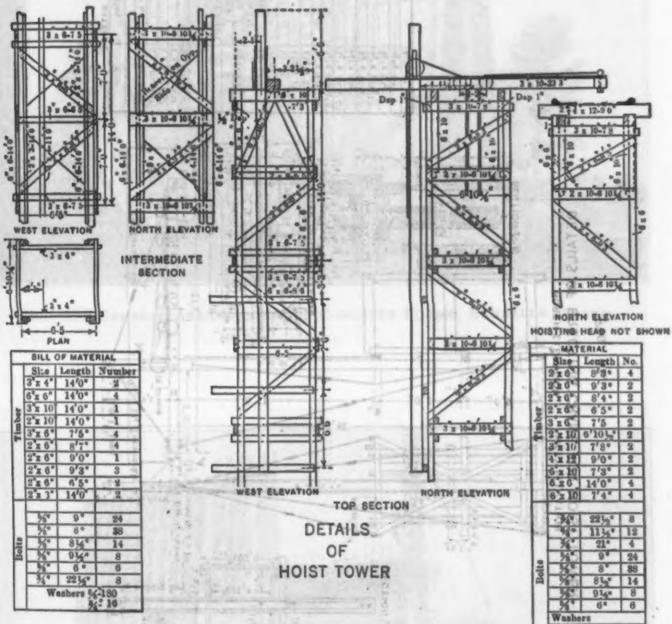


FIG. 58.—GENERAL ARRANGEMENT FOR CHUTING CONCRETE.

FIG. 59.—WARDS ISLAND UNLOADING PLANT.

At the time the contract for this masonry was undertaken, the method of chuting concrete from the mixer to its final disposition, by using a tower, had come into general use in concrete construction, and, after careful consideration of various other methods of procedure, this one was adopted as the most feasible for this work. It was proposed at first to adopt this method with four piers, but, on further consideration, it was decided to extend it to six piers as a unit. This arrangement necessitated an exceptionally tall tower (214 Mr. Seaman.



BILL OF MATERIAL			
	Size	Length	Number
Timber	3"x 4"	14'0"	2
	6"x 6"	14'0"	4
	3"x 10	14'0"	1
	2"x 10	14'0"	1
	3"x 6"	7'5"	4
	2"x 6"	8'7"	4
	2"x 6"	9'0"	1
	2"x 6"	9'3"	3
	2"x 6"	6'5"	2
	2"x 3"	14'0"	2
Bolts	5/8"	9"	24
	5/8"	8"	28
	5/8"	8½"	14
	5/8"	9½"	8
	5/8"	6"	6
	3/4"	22½"	8
	Washers 5/8"-180, 3/4" 16		

MATERIAL			
	Size	Length	No.
Timber	2"x 6"	8'3"	4
	2"x 6"	9'3"	2
	2"x 6"	8'4"	2
	2"x 6"	6'5"	2
	3 x 6"	7'5	2
	2"x 10	6'10½"	2
	3"x 10	7'8"	2
	4"x 12	9'0"	2
	6"x 10	7'3"	2
	6"x 6	14'0"	4
	6"x 10	7'4"	4
Bolts	3/4"	22½"	8
	3/4"	11½"	12
	3/4"	21"	4
	5/8"	9"	24
	5/8"	8"	88
	5/8"	8½"	14
	5/8"	9½"	8
	5/8"	6"	6
	Washers		
	3/4"		48
	5/8"		180

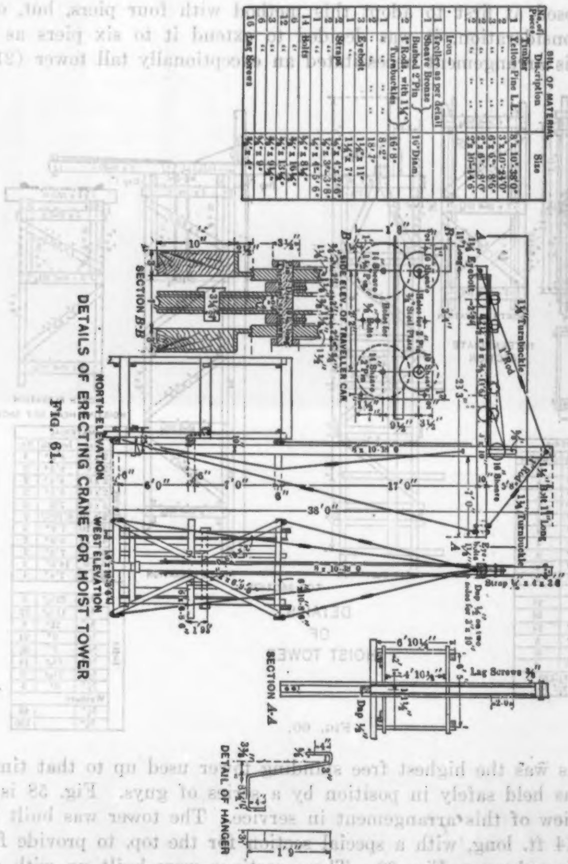
FIG. 60.

ft.). This was the highest free standing tower used up to that time, and it was held safely in position by a series of guys. Fig. 58 is a general view of this arrangement in service. The tower was built in sections 14 ft. long, with a special section for the top, to provide for the working sheaves, Fig. 60. These sections were built up with an erecting crane, Fig. 61, which was carried up with the tower as the erection proceeded, and was used as part of it while in service.

The planning of the construction of this masonry, which was in charge of the writer as Consulting Engineer for the contractor, was

Mr. Seaman.

At the time the contract for this masonry was undertaken, the method of chuting concrete from the mixer to its final disposition, by using a tower, had come into general use in concrete construction, and, after careful consideration of various other methods of procedure, this one was adopted as the most feasible for this work. It was propose[illegible] with four piers, but, on further con[illegible] extend it to six piers as a unit. This [illegible] an exceptionally tall tower (314



DETAILS OF ERECTING CRANE FOR HOIST TOWER

Fig. 61.

ft.). This was the highest free standing tower used up to that time, and it was held safely in position by a series of guys. Fig. 58 is a general view of this arrangement in service. The tower was built in sections 14 ft. long, with a special section for the top, to provide for the working sheaves, Fig. 60. These sections were built up with an erecting crane, Fig. 61, which was carried up with the tower as the erection proceeded, and was used as part of it while in service.

The planning of the construction of this masonry, which was in charge of the writer as Consulting Engineer for the contractor, was

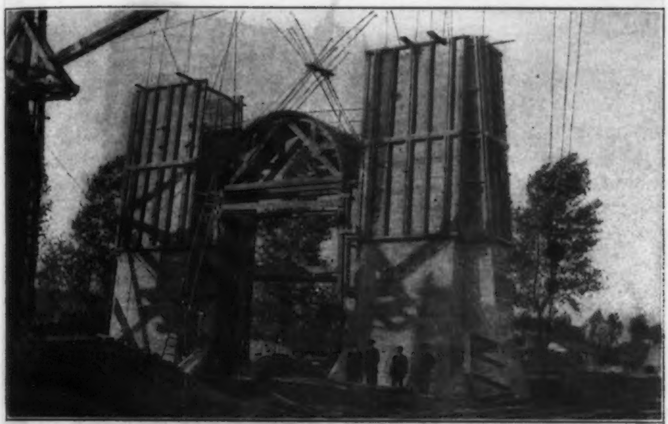


FIG. 62.—GENERAL ARRANGEMENT OF CONCRETE FORMS, RANDALLS ISLAND.

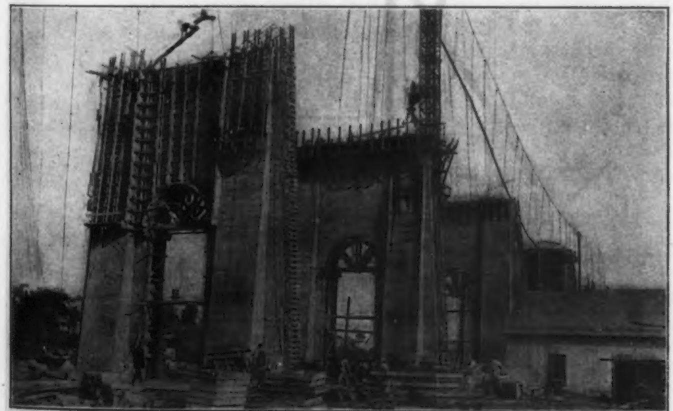


FIG. 63.—CONSTRUCTION OF CONCRETE PIERS ON LONG ISLAND.

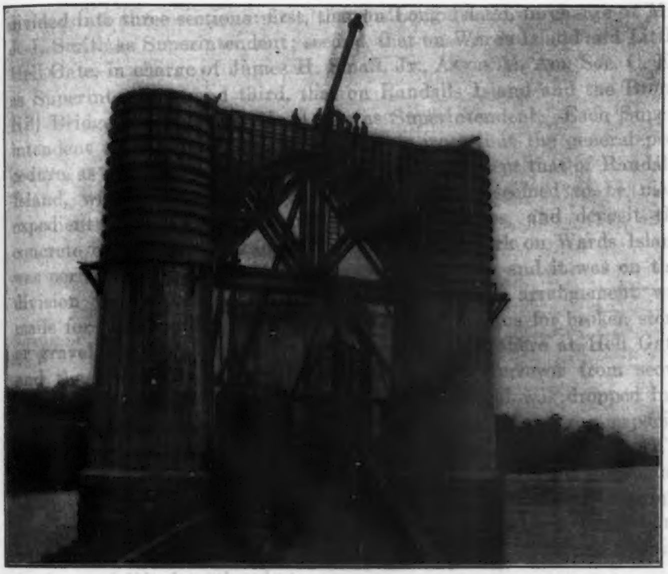


FIG. 64.—FORMS AND CENTERS IN PLACE FOR PIERS OF LITTLE HELL GATE BRIDGE.

FIG. 65.—PIERS FOR LITTLE HELL GATE BRIDGE PROTECTED BY HEAVY CRIB COFFER-DAMS.

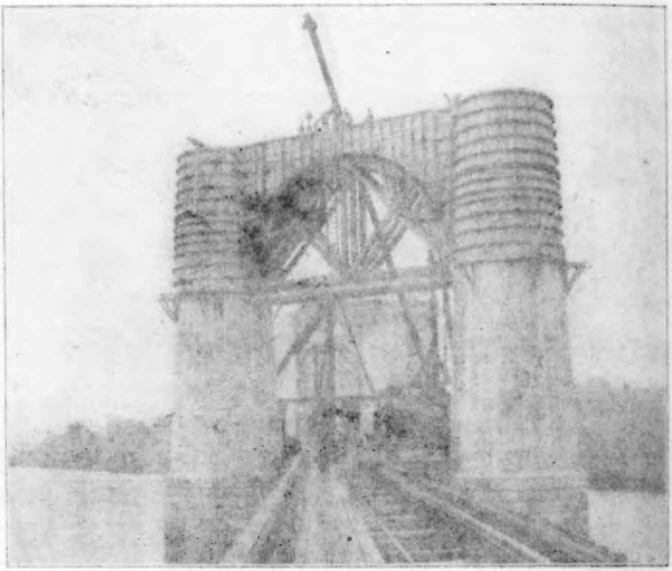


Fig. 64.—Forms and Centers in Place for Piers of Little Hell Gate Bridge.

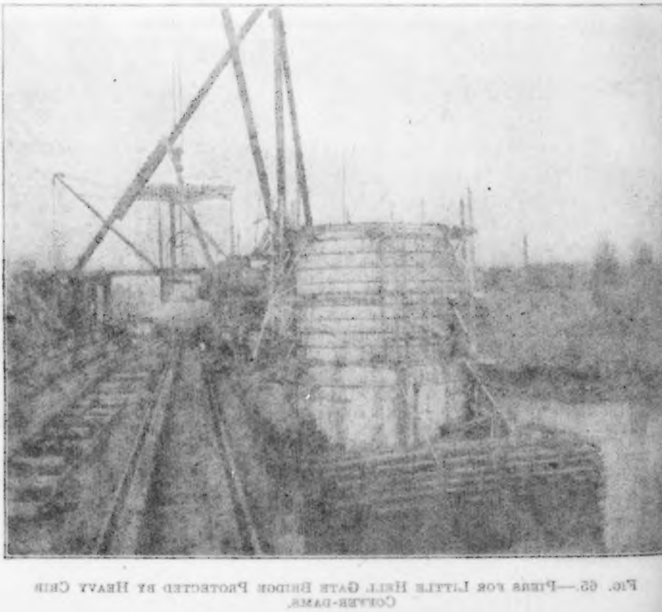


Fig. 65.—Piers for Little Hell Gate Bridge Protected by Heavy Crib Coffer-dams.

divided into three sections: first, that on Long Island, in charge of Mr. J. J. Smith as Superintendent; second, that on Wards Island and Little Hell Gate, in charge of James H. Small, Jr., Assoc. M. Am. Soc. C. E., as Superintendent; and third, that on Randalls Island and the Bronx Kill Bridge, in charge of Mr. Little as Superintendent. Each Superintendent arranged the details of his own work, but the general procedure, as outlined, was adopted on all sections, except that of Randalls Island, where the piers were lowest. There it seemed to be most expedient to place derricks on elevated platforms, and deposit the concrete by buckets, instead of by chutes. The work on Wards Island was perhaps the most carefully prepared in layout, and it was on this division that the most complete and satisfactory arrangement was made for handling the materials. A large set of bins for broken stone or gravel, sand, and cement was erected near the shore at Hell Gate, and the material was carried by a 20-in. belt conveyor from scows to the bins, Fig. 59. From these bins the material was dropped into cars especially constructed for the purpose, each having a capacity of about 12 cu. yd., and by this means conveyed to the mixer. The mixer was placed so that the charging hopper was set below the surface of the earth, thus permitting the material to be dropped in proper proportions directly into the hopper. The mixer was then charged and, after a rotation of about 1½ min., was discharged into the hoisting bucket and lifted to the chutes. Mr. Seaman.

In the design of the forms, attention was paid to the importance of duplication, and standard sections were used from the top of the pier downward, the variation in height of the piers being provided for by the height of the first form placed. The forms were in sections of about 12 by 14 ft., and were made of 2-in ship-slab sheeting, planed on one side, with 4 by 8-in. studs, 2 ft. 6 in. apart. The forms were held in place by 6 by 8-in. wales, 6 ft. apart. These forms were bolted end to end, so as to prevent leakage. Adjustable corner forms were used at the recess of the pier. The heavy batter (1:1) at the base of the pier was made by a form of special construction. The arch centers were designed so as to throw the thrust directly on the pier as a buttress, and were supported by timbers set into the concrete. These timbers were arranged so that they could be unbolted and withdrawn without injury. Considerable difficulty was experienced in removing these arch centers, and, on a repetition of the work, a modification would probably be devised by which the form could be hinged at the top, and thus made collapsible.

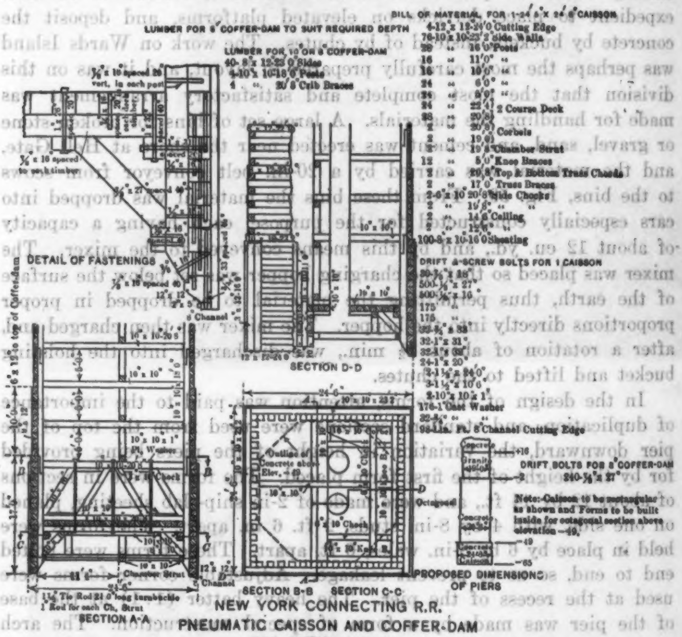
Owing to the great weight of the forms, a special crab-derrick was placed on top of the concrete and raised as the pier progressed.

Figs. 62 and 63 show the general arrangement of this work on Randalls Island and Long Island, respectively.

Mr. Seaman.

Fig. 64 shows the construction of the forms and centers for the piers of Little Hell Gate Bridge. The forms are practically hooped barrels, the first form being of variable height, as already described for the rectangular piers.

The foundations of these piers are on rock, about 12 ft. below low water, and in a current running 7 miles per hour. The construction was protected by heavy crib coffer-dams (Fig. 65). The pit was pumped dry and the bottom leveled off. Forms were then set inside for the



NEW YORK CONNECTING R.R.
PNEUMATIC CAISSON AND COFFER-DAM
FOR PIERS 20 & 21
BRONX KILLS BRIDGE
FIG. 66.

concrete. On this concrete foundation, masonry was laid, about 12 ft. high, so as to protect the pier at the water surface. On this masonry the concrete shafts were then constructed. A trestle of simple pile construction, three piles per bent, with 12 by 12-in. caps and 12 by 12-in. stringers, was used at this river work.

The construction of the tower of the Wards Island foundation for the main arch span has already been described by Mr. Ammann. The preliminary studies of this work on the part of the contractor

contemplated sinking these various cylindrical and rectangular concrete caissons by open dredging. This process proceeded with eminent satisfaction as to accuracy to a depth of about 30 ft. below ground, when it was believed by the engineers that greater progress could be accomplished by the use of compressed air. As such work, however, had not been contemplated by the contractor, it was taken over by the Railroad Company. Mr. Seaman.

The foundations of the Bronx Kill Bridge were sub-contracted to the Arthur McMullen Company. Fig. 66 shows the caissons used by that firm. The deepest part of the foundations for these piers was 91 ft. below high water.

GUSTAV LINDENTHAL,* M. AM. SOC. C. E. (by letter).—Mr. Breithaupt believes that for Little Hell Gate either spandrel-braced or double-ribbed arches should have been used, in place of the inverted bow-string type. Mr. Lindenthal.

There is no navigation on this short stream, except that of small pleasure launches and sailing boats. The clear height above the water was of less importance, in the opinion of the Government engineers, than the obstruction from piers. It is expected that this channel (which connects the East River with the Harlem River) will be deepened, not for purposes of navigation, but for a more ample flow of the tides through it. Long piers in the direction of the currents were regarded as offering more obstruction to the tidal flow than the column piers as arranged. Metal arches in place of bow-string girders would have required heavier column piers to take up the resultant between loaded and unloaded adjacent spans. There would have been, not only no saving in the total cost of this crossing, but more obstruction to the current.

For an arrangement of four continuous spandrel-braced arches, delivering only vertical pressure on the river piers and subject to heavy temperature stresses, violent reversion stresses, and large secondary stresses, the relation of live load to dead load was not favorable. It is not an arrangement suitable to a railroad bridge.

At first, parallel-chord, riveted trusses were considered, but most of the members were so large as to require cross-sections with four webs. That is very objectionable in tension chords. The inverted bow-string girder, with the tension chord composed of eye-bars, made a more satisfactory structural arrangement. The web stresses in this form of truss are small, and the details for them simple. The eye-bars are the heaviest ever forged—16 by 2¼ in., with 16-in. pins.

The superstructure receives its lateral rigidity through an extra heavy and rigid wind and vibration truss in the plane of the top chords. It is sufficient to transmit the wind pressure, which acts on the bottom

* New York City.

Mr. Lindenthal.

chord at each vertical web member, by a stiff diagonal to the wind truss above; therefore, no bracing is required in the bottom chord. The correctness of this detail is proved, also, by the fact that the bridge is laterally quite rigid under trains. The rigidity of the superstructure is further enhanced by the fact that its center line of gravity is about 15 ft. below the points of support on top of the piers and abutments. It would be in stable equilibrium, therefore, even without any lateral bracing below the wind truss.

It will be noticed, in the silhouette of the bridge, that its floor-table has the same thickness (10 ft.) as the plate-girder approaches. The harmony of design is thus preserved. That uniformly deep floor-table forms a continuous heavy line on top of the masonry piers; it appears on this structure, at a distance, as if supported on slim catenary chains, but, in fact, these chains each form the heaviest eye-bar chord member in any existing, simple-span, truss bridge.

The masonry tower piers at each end of Little Hell Gate form a sufficient break to mark the longer spans. These towers are hollow, and contain emergency stairways. Their form, it is true, could have been somewhat improved, had they been studied in a model, as was done in the case of the granite towers for the large arch span.

The writer would have preferred to make provision from the start for a second deck throughout for a boulevard over the railroad tracks, which he foresaw would be wanted in the future for highway traffic between the Bronx and Long Island; but his efforts at that time to get the public and city authorities interested in a boulevard were not successful, and were also not favored by the Railroad Company. The time will come, however, when such a boulevard may be added. It will be cheaper to do that than to build a separate parallel structure for that purpose.

Contrary to Mr. Quimby's opinion, the span length of the concrete viaduct is not too short. As stated in the paper, the most economical length was chosen, except where affected by the location of the streets. By examining the costs of masonry and steelwork given, Mr. Quimby can easily convince himself of this fact.

An important reason for adopting the concrete piers was that objection was made by the authorities of Wards and Randalls Islands to the steel columns, because they feared that inmates of the municipal institutions on those islands would climb them and make their escape. It was insisted that the design adopted should prevent this.

Mr. Quimby's idea as to what should have been the arrangement of the end panel in the large arch is based on an erroneous assumption. Assuming that the top chord of the arch would be anchored in the towers, making of the arch a type with fixed ends and end moments, a diagonal member to the end of the top chord would be entirely superfluous, and would appear so to the technically trained eye, as the

shearing stress would be taken care of by the masonry. The anchoring of the top chord, however, would have required much heavier stone towers, and of different form, in order to take up the horizontal anchorage stresses at the top. Mr. Lindenthal.

The top chord was prolonged into the masonry simply for convenience of inspection. There is a little stairway hidden in the end of the deep top chord over which one is enabled to enter from the interior of the tower and go out on the chord. The objection of the Municipal Art Commission to the first design was merely against the too ornate form of the towers of moulded concrete. The form of towers, therefore, was simplified, and granite was chosen for the exterior faces.

CLEMENT E. CHASE,* JUN. AM. SOC. C. E. (by letter).—It is largely through innovations introduced into the design of large bridges, such as the Hell Gate Arch, that structural engineering advances. Precedents launched with such prestige may, in fact, become established in practice without being subjected to such critical examination of their merits as should be the case. An important new feature of the Hell Gate design, quite likely to be of wide influence, is the use of single, unusually heavy plates for the webs of the main chords. Mr. Chase.

Mr. Ammann states:

"It is frequently maintained that better material is obtained in the thinner than in the thicker plates. This was not substantiated by the great number of specimen tests made for the Hell Gate Bridge from material varying from ½ in. to 2 in. in thickness. In general, the thick material showed as high elastic properties and ultimate strength as the thinner material rolled from the same heat."

From this the impression might be gathered that, in some unexplained way, the mills which produced the steel for the Hell Gate Arch had overcome the difficulties which had hitherto prevented the thickness of material being increased beyond average gauges (⅜ to ⅝ in.) except at the expense of quality. This decrease in quality with increase in thickness has not only been the commonly accepted belief of those whose experience has been greatest in testing steel, but there is on record ample experimental evidence to support this view, for example, Table 16 in the paper by Henry S. Prichard, M. Am. Soc. C. E., entitled "The Effects of Straining Structural Steel and Wrought Iron."†

It would seem that Mr. Ammann relies on the yield point and ultimate strength showing of the specimen tests (and these, presumably, the inspector's mill tests) to support the belief that the thick plates were obtained at no sacrifice of quality. However, neither of these values, as commercially determined, can be accepted as more than

* Poughkeepsie, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 1541.

Mr. Chase. a fractional part of the measure of quality. For its complete determination, the actual elastic behavior, such as is presented to the eye in an accurate stress-strain curve, and the ductility, at least, must be known. Quality for structural uses should also take account—though possibly in lesser degree—of shock resistance and endurance under repeated loadings.

The increasing inferiority in ductility with increasing thickness is usually taken note of in specifications, to which the Hell Gate requirements (Table 1) are no exception.

There is no mystery in the inverse relation of thickness and quality of rolled steel; the necessities of commercial mill practice are sufficient explanation. Usually, plates or shapes of a wide range in weight per foot are rolled from ingots of the same cross-section, which causes the ratio of reduction to be less as the thickness of the product increases. Aside from this, the construction of the mills in use, which were designed almost without exception for lighter sections, demands that, when heavy products are being rolled, the steel must be finished hotter. Even if finished at the same temperature, a thick piece will cool more slowly than a thin one. The result, considered in terms of micro-structure, will be a coarser-grained material. It is well established that the larger the grain size of steel, the lower will be the stress at which deviation from elastic behavior occurs, the lower the elongation and reduction of area at rupture, the lower the shock resistance and the endurance under repeated loads.

The yield point and ultimate strength determinations of the mill tests, on which Mr. Ammann based his statement, have their usefulness in guarding against serious variations of a standard product from normal, but, as experimental proof of the adequacy of a new product for a new use, they would have but little weight. There are several reasons for this: they are made hurriedly, in commercial routine, which must disregard the niceties of testing; the samples selected are not representative of all parts of the material; and, as explained before, these characteristics are themselves a very incomplete statement of quality.

Ordinary mill tensile tests are completed at a rate of from thirty to fifty per hour, and, even under the severest specifications in use, they are made much too quickly to have any scientific standing. It is not the writer's understanding that the Hell Gate specimens were tested with any refinements beyond those customary in inspecting material for other high-grade bridgework.

The misrepresentative character of mill tests is a matter of common knowledge. In comparing heats rolled into similar products, this may be overlooked, for the specimens are taken from the same locality in the product each time, and the tests are comparable, as far as they go. To judge of the full strength of the material over its entire

cross-section, they would be inadequate. For instance, in the case of universal mill plate, tests are cut next to the rolled edge. By reason of the amount of work received, the lower working temperature, the radiation from three sides in cooling, and the distance from the more or less segregated axis of the plate, this part of the plate is ordinarily much superior to any other. It will show a higher elastic ratio and higher ductility than the center, and the thicker the plate the greater the difference between the showing of the edge and what would be the showing of the center. For sheared plate, the circumstances are much the same, except that the actual ragged edge is first removed by shearing. Mr. Chase.

That the yield point and ultimate strength (even if accurately determined and representative) do not alone tell the whole story of the structural usefulness of metal has, for columns, been strikingly pointed out in the recent final report of the Society's Special Committee on Steel Columns and Struts.

It is stated therein that:

"Neither the 'yield points' as indicated by the drop of the beam, nor the ultimate strengths, bear any consistent relation to the U. L. P.'s, or ultimate strengths, of the full-size columns."

The point to which is ascribed the greatest importance, the "Useful Limit Point", can only be located when the critical portion of the stress-strain curve is known with precision. Barring accidental blows, which would call ductility and shock resistance into play, or unlikely repetition of over-loads to the extent of testing repeated stress endurance, it is this curve which determines structural fitness. This is as true of tension members as of the compression members with which the report deals.

Bearing directly on the subject under discussion, the Committee finds that the useful limit of a column's unit strength decreases as the thickness of the component material increases.* However, the Committee's tests did not include columns of equal cross-section, made up in the one case of single, thick plates, and in the other of several thinner plates, stitch-riveted together, and this leaves a loophole for arguments such as Mr. Ammann's on page 891. Referring to the Committee's test, he says:

"If the heavy columns had been built up of several thin plates, tack-riveted together, their compressive strength would probably have been even less."

Although, possibly, some weakness is introduced in this way, the writer feels that it is of far smaller degree than the influence of thickness of material in reducing the limit of structural usefulness

* The reader is referred to Tables 22 and 23 of the Final Report of the Special Committee on Steel Columns and Struts, *Proceedings*, Am. Soc. C. E., December, 1917.

Mr. Chase. under load. It is rather strange that, with all the large-sized column tests which have recently been made, there should have been none to throw light on this point directly. The designers of the new Quebec Bridge evidently held one view, the designer of the Hell Gate Arch, the other; and yet only a few, relatively inexpensive tests would be required to settle the matter. The question is of fundamental importance, and should be removed from the field of speculation and controversy.

Mr. Ammann. O. H. AMMANN,* M. AM. SOC. C. E. (by letter).—It is gratifying to note that the design of the Hell Gate Bridge structure as a whole as well as its general structural and architectural features are approved without exception by those who discussed this paper.

Some of the criticism on the general design of the Hell Gate Arch, the Little Hell Gate Bridge and on certain features of the approach viaducts has been answered by the designer himself, and it remains only to reply briefly to the discussion of various special features and details.

Selection of Type of Bridge.—Mr. Moisseiff is of the impression that the suspension type of bridge with a longer span would have been selected had the unfavorable foundation conditions on the Wards Island side been better known. As stated in the paper, the arch was selected in preference to the suspension type principally on account of the local conditions, which precluded long shore spans. Whether a suspension bridge with a shorter span would have been more economical under the soil conditions as actually found could be determined only by a complete design, but it is unquestionable that a suspension bridge with a longer span than the arch would have been more costly, in spite of cheaper foundations.

Had the unfavorable soil conditions been fully known earlier, a relocation of the line might have been closely investigated, but it is doubtful whether any saving could have been effected, because any other location would have meant a longer span.

Towers and Foundations.—The writer agrees with Mr. Moisseiff that towers are not an economical feature if intended for the exclusive purpose of keeping the resultant reaction within the middle third of the foundation area, because, if it is a mere question of sufficient weight of masonry, such weight can be provided more cheaply by extending the foundation in the direction of the axis of the bridge. Where it is a question of providing towers for architectural reasons, however, such towers constitute at the same time a considerable saving in the foundation masonry and are then not entirely for architectural purposes.

* South Amboy, N. J.

Bottom Chord Section.—Mr. Fowler comments on the section of the bottom chord as lacking in economy, when compared with the cellular circular type. This is true as far as economy of metal is concerned, but practical difficulties in making the circular section offset the economy. The form of section which Mr. Fowler proposes for the bottom chord of a cantilever bridge of 2 000-ft. span is an excellent substitute for the circular one, and would be particularly suited also for the main rib of a great arch. It is compact, and has no outstanding flanges or wide thin webs, which are always a source of local buckling; and the effective section is well distributed at equidistant points from the center of gravity. However, on account of its many pockets and oblique angles, which make fabrication expensive, it can hardly be recommended for sections of less than 1 000 sq. in. of effective area, especially when the chord is to be tapered, as was found desirable in the case of the bottom chord of the Hell Gate Bridge. Mr. Ammann.

The Three-Faced Joint.—The comment on this joint of the bottom chord has been favorable. Mr. Moisseiff would deduct the excess stress, caused by the bearing over the middle third of the joint, from the specified unit stress for compression members. This does not appear to be necessary, and would offset the economy of the design. The bearing stress at the middle of the joint can reach the yield point without harm, as the metal thus strained is well confined on all sides within metal which is strained to less than the permissible limit, and to which it imparts its excess stress within a short distance of the joint. The three-faced joint approaches the condition of a roller, or ball bearing, for which the maximum bearing stress is allowed to run at least 50% higher than the average stress of a compression member.

Thick vs. Thin Plates.—One of the features most widely commented on is the extraordinary thickness of 2 in. adopted for the webs of the heavy compression chords. It is variously stated that experience has shown that thick metal is commonly not as strong and elastic as thin metal rolled from the same material.

The designer fully recognized this fact, and for this reason allowed a wider variation for the ultimate strength of hard steel, of which the 2-in. plates are made, than is allowed on ordinary structural steel (see foot-note below Table 1). It was expected that the thicker material would generally be below the "desired" strength or nearer the minimum, and the thinner one nearer the upper limit; also, a graduated lower limit was set for the elongation of thick material (see foot-note below Table 1).

The final specifications were drawn after thorough consultation with the manufacturers, whose obligation it was to produce, and who did produce, the desired qualities.

Mr. Ammann.

In spite of the expected slightly inferior physical qualities, the designer considered thick plates for compression members superior to thin plates, which are weakened by many stitch-rivets and the possibility of local buckling between the rivets.

The expectation of lower results for thick plates, however, was not borne out by the results. The individual test reports are not readily available at the present time, also space would not permit of their complete reproduction, but the fact is, as stated in the paper, that the tests of thick metal were generally as favorable as those of thin metal from the same heat. The only exception was the elongation, which ran slightly lower in the thick metal, but not to the extent allowed by the specifications.

The question as to the superiority of thick plates over stitch-riveted plates for compression members can be convincingly solved only by a series of systematic tests. In the absence of such tests, only judgment can govern; it would seem to the writer that it is sounder judgment to build up a member of solid metal unweakened by forceful injuries. Mr. Chase seems to take into consideration only the direct strains. How important the shearing stresses are in compression members is now well recognized with respect to the latticing of columns. That a lamellar bar is stronger in that respect than a solid bar does not seem plausible.

There is also a practical question to be considered. Rivets are apt to be less perfect the longer their grip and the greater the number of plates to be riveted together, and, therefore, it would seem that it is especially desirable to avoid riveting in large members, as far as possible.

Mr. Chase appears to assume that the writer disregarded the ductility of the steel, in comparing the test results of thin and thick metal. On the contrary, he took the elongation and reduction fully into consideration. These qualities in connection with the elastic limit and ultimate strength give the engineer a fair measure of the quality of the stress for practical purposes, without going into the theoretical niceties of stress-strain diagrams. Where the number of tests is as great as in the case of the Hell Gate Bridge, the fact that they are not made with extreme refinement has little bearing on the relative quality of thin and thick metal. Contrary to Mr. Chase's understanding, the tests were conducted with more than ordinary care and watchfulness on the part of the inspector. The testing machine was not allowed to run with a greater speed than 2 in. per min., and the objectionable features of commercial mill testing, as mentioned by Mr. Chase, were avoided. More specimens were made from each heat than in ordinary practice, and the specimens were cut both parallel and at right angles to the direction of rolling, and from both the edge and center of the plate.

Mr. Ammann.

The writer cannot see what shock resistance and endurance under repeated loads have to do with the question under consideration.

Assembling of Trusses.—In reply to Mr. Wagner's inquiry, relative to the assembling of the arch trusses, the writer wishes to state that the specifications prescribed complete assembling of the truss. On request, the Bridge Company was subsequently allowed to assemble each section of four panels, the last panel being reassembled to the next section, thus insuring accuracy for all connections.

Floor Construction.—The writer cannot agree with Mr. Wagner that the floor construction would have been made more durable by water-proofing, by which presumably is meant a water-proof covering of the concrete. The cost of a good water-proof covering is out of proportion to the advantage gained, but even the best water-proofing known cannot be as durable as the concrete base itself. If it becomes defective, it is likely to cause more trouble than if it had not been used. In the case of the Hell Gate Bridge and approaches, the concrete slab has been made water-proof by carefully selecting the materials and determining the proper proportion to secure the greatest density, by carefully troweling the surface to a smooth finish before the concrete had set, by using sufficient steel reinforcement to prevent cracking, and by giving the surface sufficient slope and providing frequent and large drain holes to give the water a free run-off.

In concluding, the writer heartily thanks all who have taken the pains to discuss this paper for their courteous expressions of approval of the manner in which this rather complex subject has been presented. He earnestly hopes that the discussion on some of the disputed questions may be an inducement for further investigation.

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Paper No. 1418

STRESS MEASUREMENTS ON THE HELL GATE ARCH BRIDGE*

BY D. B. STEINMAN, ASSOC. M. AM. SOC. C. E.†

WITH DISCUSSION BY MESSRS. H. J. BINGHAM POWELL, J. A. L. WADDELL, F. H. FRANKLAND, L. A. WATERBURY, F. D. HUGHES, A. H. FULLER, JAMES E. HOWARD, ISIDORE DELSON, GUSTAV LINDENTHAL, JOHN I. PARCEL, H. M. MACKAY, F. E. TURNEAURE, HENRY S. JACOBY, O. H. AMMANN, C. A. RANDORF, DAVID A. MOLITOR, AND D. B. STEINMAN.

SYNOPSIS.

The magnitude of the Hell Gate Arch Bridge and its interesting construction features suggested the desirability of utilizing the bridge as an instrument for scientific research by conducting a series of stress measurements extending through all the different stages of erection until the structure was completed. As a result of these observations, the Hell Gate Arch is probably the only structure ever built in which the true stress conditions are known from experimental determination.

In order to give this investigation its proper setting and perspective, there is presented, as an introduction, a résumé of previously published measurements of stresses in bridges, with a brief statement of the most significant results obtained. From these findings, particularly with reference to secondary stresses, conclusions are drawn by which to improve the design of railway viaducts and bridges. The record, on the whole, indicates the scarcity of such observations in the past, and serves to emphasize the value and importance of undertaking

* Presented at the meeting of November 21st, 1917.

† Now M. Am. Soc. C. E.

more investigations of this character in order to confirm or correct the results of theoretical analysis. This experimental verification of stress conditions is particularly desirable in fields previously unexplored, or wherever there are special features which may produce uncertain variations from calculated conditions.

The original objects in view in undertaking these stress measurements were: to follow up the stresses in arch and back-stays as critical erection stages were approached; to check and control certain critical operations in the erection; to check the stresses in the completed indeterminate structure, in order to detect any variation from assumed conditions; and to determine the true secondary stresses for comparison with the calculated values.

The paper includes a brief description of the instruments, the method of operation, and the precautions required for accurate work. This is followed by a full presentation of the results, including a tabulated comparison of the calculated and measured primary and secondary stresses.

From the results of the measurements and comparisons, conclusions are deduced on the following subjects:

1.—The precision attainable with the instrument, and the reliability of this method of measuring stresses;

2.—The probable error of calculated stresses, indicating the futility of excessive precision in their computation;

3.—The extreme variations of fiber stress in a member, representing the combined effect of all known and unknown secondary strains;

4.—Safe working values for the design of bridge members, leaving a margin below the elastic limit for extreme variations from calculated stresses;

5.—Relative values of calculated and measured secondary stresses (it is shown that the latter are generally lower);

6.—The effect of the erection operations on the secondary stresses;

7.—The efficacy of the three-faced joints in the lower chord (Fig. 6), which were provided in order to produce a hinge action at the panel points during erection and to concentrate a larger part of the direct stress in the middle third of the cross-section, it having been demonstrated that this novel splicing feature has accomplished the desired objects, thereby reducing the secondary and direct stresses in the outer fibers;

8.—The re-distribution of stresses and the release of secondary strains at a splice during the replacing of drift-pins by rivets;

9.—The comparative freedom of the arch truss from secondary stresses; and

10.—The final extreme fiber stresses as compared with the calculated values.

Special acknowledgment is due to Gustav Lindenthal, M. Am. Soc. C. E., the Chief Engineer of the bridge, who undertook to make these measurements in furtherance of engineering science.

Résumé of Previously Published Stress Measurements.

Since the early days of bridge building, when empirical methods of design prevailed, theoretical analysis has made rapid advances, and has now far outstripped experimental verification. There is a growing movement, however, as shown by the modern testing of large compression members, and in load tests and strain measurements on bridges, to supplement the results of analysis with experimental observation in order to confirm or correct the former, or to reveal unsuspected conditions.

The first measurements of the actual stresses in trusses were made by Professor Fränkel* in 1883. He applied his extensometer to the experimental determination of secondary stresses, and his results produced such uneasiness among German engineers that some of them abandoned the riveted truss in favor of pin connections.

In 1899 Mesnager published an account of the measurement of stresses in a bridge of 180 ft. span, on the Orléans Railway in France, with a discussion† of the results. It was a ten-panel, Pratt truss bridge carrying a single track. The stresses were measured only in the web members, and for a train load of two locomotives and four cars, covering the entire span. The extensometers, devised by M. Rabut, were applied at four points of the cross-section near each end of a member. The results showed the average stresses in the members to be in fair agreement with the calculated primary stresses. There were considerable differences, however, between the stresses on opposite sides of a member. In the posts, secondary stresses were found amounting to 200% of the direct stress, involving a complete reversal

* "Versuche mit dem Dehnungszeichner", *Der Civilingenieur,* 1883.

† "Les Fatigues Réelles et Fatigues Calculées dans un Pont à Grandes Mailles", *Annales des Ponts et Chaussées,* 1899, II.

of stress on one side of the member. In the diagonals the greatest secondary stresses were 45% of the direct stress. The foregoing large secondary stresses do not form the basis for any general conclusions, as they were mainly due to peculiarities in the design of the structure.

Mesnager did not undertake a computation of the secondary stresses in the test bridge, so that no comparison of actual with calculated values is afforded.

In 1901 M. Rabut described* a series of stress measurements which had been made on the bridges of the Orléans Railway. The experiments covered small plate-girder and pony-truss spans. In some plate girders, in reference to which apprehension had been aroused by excessive calculated stresses, the measured stresses were found to be considerably lower. Plate girders supporting longitudinal ties and rails were found to act as beams with fixed ends, with a large reduction of stress at the center, and reversed stresses at the ends. In pony trusses, secondary stresses as high as 30% appeared on the outside of the top chords, caused by the trusses bending inward near the center of the bridge as a result of the floor-beam deflections. The bottom chords of through bridges showed smaller stresses than the top chords, on account of the former being partly relieved by the stringers and the bottom laterals taking part in the elongation. All floor-beams were found to act nearly as simple beams, the end constraint being negligible. Stringers acted as simple beams for a load on one side of the floor-beam, and as beams with fixed ends for symmetrical loading.

The results of these experiments demonstrated the necessity of detailing stringers for full negative bending moments where they frame into floor-beams; the desirability of using deep stringers to avoid high torsional stresses in the floor-beams from one-sided loading; and the importance of using deep floor-beams and verticals comparatively slender in the transverse direction in order to minimize the bending stresses arising from the floor-beam deflections.

In 1905 and 1906, W. Gehler conducted a series of tests and measurements† on a railway bridge of 128 ft. span at Elsterwerda, Saxony. It was a ten-panel, Pratt truss, skew-bridge; and the loading

* "Conference sur l'Experimentation des Ponts", *Annales des Ponts et Chaussées*, 1901, III.

† "Nebenspannungen eiserner Fachwerkbrücken", Berlin, 1910.

consisted of two tank cars placed near the middle of the span. The vertical deflections at the lower panel points were measured with Fränkel-Leuner deflectometers, and the rotation angles of the same points were observed with extreme care by using highly sensitive spirit levels. The measurements agreed closely with the values previously calculated, except for some small variations which were traced to the stiffness of the track and the effect of the skew arrangement of the trusses. The results, on the whole, afforded a valuable proof of the remarkably close agreement attainable between theoretical computations and actual conditions in a bridge structure.

Gehler had computed the secondary stresses in the trusses from the theoretical vertical and angular deflections of the lower panel points; and he regarded this check on these deflections as a verification of the calculated secondary stresses. He recommends this procedure as an experimental method for determining the actual secondary stresses in a truss.

To make the foregoing investigation complete, some direct stress measurements were made with Fränkel-Leuner extensometers,* which give automatic graphic records of the varying stresses. On account of the unsatisfactory precision of these instruments, the results were so erratic that Gehler concluded that extensometers are not suitable for the measurement of secondary stresses.

In 1907-09 a sub-committee of the American Railway Engineering Association, consisting of F. E. Turneaure, M. Am. Soc. C. E., C. L. Crandall, M. Am. Soc. C. E., the late C. H. Cartlidge, M. Am. Soc. C. E., and the late C. C. Schneider, Past-President, Am. Soc. C. E., conducted a series of tests on a large number of plate-girder and truss bridges, ranging in span from 30 to 440 ft. They used special test trains, and measured the deflection at the center of the span and the strains in the various kinds of members. The object of the tests was, not to determine secondary stresses, nor to compare calculated and measured primary stresses, but to ascertain the relative amounts of the resulting deflections and strains for various speeds of the moving loads, in order to establish the proper provision to be made for impact stresses. The results and conclusions are contained in the report of the Committee.†

* *Der Civilingenieur*, 1882, p. 200.

† *Bulletin No. 125*, Am. Ry. Eng. Assoc., July, 1910.

In 1911 the Sub-Committee of the American Railway Engineering Association made a theoretical and experimental study of secondary stresses in truss bridges. In several bridges, of both riveted and pin-connected types, the secondary stresses at selected points were measured with extensometers. In the case of a 105-ft. pony Warren truss, a comparison was made of the calculated and measured secondary stresses in the members. The results are published in the report* of the Committee.

With the exception of the 105-ft. span last mentioned, the writer knows of no published comparison between measured and calculated secondary stresses.

Gehler's work was only a partial check on the calculations of secondary stresses, as he merely measured functions, namely, panel point deflections and rotations, from which the actual secondary stresses had to be computed.

All the published stress measurements were made for loads on existing structures. They afford no information as to the strains produced by dead load or the effect of the erection on the secondary stresses.

None of the published measurements was made on indeterminate structures or on any structure exceeding 440 ft. in span.

Reasons for Taking the Measurements.

The magnitude of the Hell Gate Arch Bridge and its unusual structural features suggested to Mr. Lindenthal the desirability of utilizing the structure for scientific research. By preparing the members for extensometer measurements and taking initial readings before erection, the huge arch was converted into an instrument for the experimental study of the true stress conditions in a structure.

The trusses of the bridge are doubly indeterminate: they are two-hinged arches, and they have redundant diagonals in the center panel. It has often been advanced, as an objection to the use of indeterminate bridge types, that the computations are highly theoretical and the actual stress conditions are uncertain. It is hardly necessary to remark that this argument should have little weight under modern methods of design and construction. As an answer to such objections in the future, it appeared to be of interest to demonstrate, by measure-

* *Bulletin No. 163, January, 1914.*

ment, the identity of calculated and actual stresses in the finished structure.

Another possible purpose of extensometer measurements is to supply an added safeguard for a structure during erection by measuring the stresses at critical points. The stresses in the various members of the arch and in the back-stays were measured in the successive stages as the trusses were built out from shore, panel by panel, to their junction at mid-span. When any of the stresses were nearing their maximum values, they were closely watched until the critical stage was passed.

In one instance, one of the eye-bars in the forward stay was found to have a stress 55% higher than the average stress in the group of eye-bars. This appeared in an early stage of the erection, when the stresses were low. As the erection progressed, however, and as was to be expected, the difference in stress gradually diminished until, in the last stage, all the eye-bars were found to be uniformly strained to nearly 20 000 lb. per sq. in.

Another application of the extensometer was to provide a check in some of the critical operations in the erection of the structure. After seven panels had been erected, the hydraulic jacks at the tops of the erection towers were operated to put the forward stay in tension and release the lower stay. The completion of this operation was checked by measurements of the stresses in both stays. When this jacking was performed on the Wards Island side, a small compression was found in the lower stay, indicating that the jacking had gone too far; the jacks were then lowered slightly until the stress was reduced to zero.

After the trusses met at mid-span, the lowering of the jacks to close the arch was closely followed by measurement of the diminishing stress in the forward stay. Measurements were also made in the central chord member, in order to make sure that there was no eccentricity of bearing at the junction of the two half-arches.

The final operation consisted in closing the top chord at the crown in order to convert the structure into a two-hinged arch. A large bolt, connecting across the joint, was provided in order to hold the members together until the drilling of holes and riveting could be completed. This bolt had to be adjusted, by a set of nuts, to an initial condition corresponding to zero stress at 60° Fahr. The exten-

someter was used to control this operation and to check the subsequent stresses in the top-chord member.

The final object of the extensometer measurements was to provide a comparison between the calculated and the actual secondary stresses in the structure. In addition to the general scientific value of such a comparison, there were special conditions which called for a determination of the true stresses. These conditions were the cantilever method of erection, the use of drift-pins for temporary connection and their subsequent replacement by rivets, the three-faced butt-joints in the lower chord, and the unprecedented dimensions and form of cross-section. These features, separately and combined, modify the secondary stresses materially, and render it extremely difficult, if not impossible, to arrive at the true secondary stresses by calculation.

THE INSTRUMENT AND THE METHOD OF ITS OPERATION.

The instrument used for the stress measurements on the Hell Gate Arch Bridge (Fig. 1) was a 20-in. strain gauge, designed by Mr. James E. Howard. This instrument is essentially a micrometer caliper. The measuring points, made of hard steel and conical in form, are attached to the barrel and the rod, respectively; and the distance between the two points is measured by a micrometer contact screw at one end of the barrel. This screw reads, by a circumferential vernier, to 0.0001 in. It is provided with two milled heads having a spring ratchet between them which slips when contact is made. Some operators prefer to use the inside milled head, relying on their sensitiveness of touch to detect the instant of contact; others use the outside or ratchet head. The barrel of the instrument is covered with leather for mechanical and thermal protection.

Accompanying the instrument is a rectangular steel bar used as a reference or comparison bar (Fig. 2). It is provided with two center holes or gauge points 20 in. apart. Whenever a reading is taken on the member, a comparison reading is taken on the bar; the difference between these two readings is the measurement. This method of operation eliminates the effects of personal equation, as the same observer takes the readings on the bar and on the member. It also dispenses with the necessity of knowing the temperature of the instrument, as the unknown effect of this temperature is eliminated in subtracting the member reading from the bar reading.

No expansion correction is necessary if the reference bar and the member have the same temperature; and, for this reason, the bar is allowed to rest on the member in order to equalize the temperatures before the extensometer is applied. Nevertheless, two thermometers are provided, one permanently inserted in the face of the reference bar and the other to be placed between the gauge points on the member being measured. When the measurement is taken, the two thermometer readings are observed and recorded, so that a correction may be made for any difference between them.

There is also furnished with the instrument a steel trammel bar (Fig. 3) holding two prick punches 20 in. apart. This is used for laying out the holes in the members to be tested.

The holes in the steel are drilled with a hand ratchet drill provided with combination bits, consisting of drill and countersink. The size of the drill is No. 57, and the countersink is 60 degrees. After drilling a hole, the edge between the countersink and cylindrical shaft is removed with a center punch which has been ground to an included angle of 55 degrees. With the tap of a hammer, the punch leaves a small conical seat to receive the measuring points of the extensometer which are also ground to 55 degrees. This special form of recessed gauge mark, with its combination of three surfaces, has a number of advantages: The extreme tips of the measuring points are not used, thus eliminating the errors which would result from the unavoidable wear of these tips; the contact is on an area, instead of a point or line, thereby reducing the wear of the measuring points and the errors from compression of the bearing surface; the bearing area is depressed below the surface of the steel, so that it is better protected from surface dirt and mechanical injury. The 60° countersink helps in guiding the measuring points to their seat. The cylindrical shaft will hold small particles of dust or grit remaining at the bottom of the hole without interfering with the precision of the measurement.

Oil, vaseline, or ivory black paint are used for protecting, filling, or covering the holes, depending on the interval between successive measurements. A pointed aluminum rod, a small cedarwood stick, and some absorbent cotton are used for cleaning the holes before taking any readings.

FIG. 1.—HOWARD EXTENSOMETER.

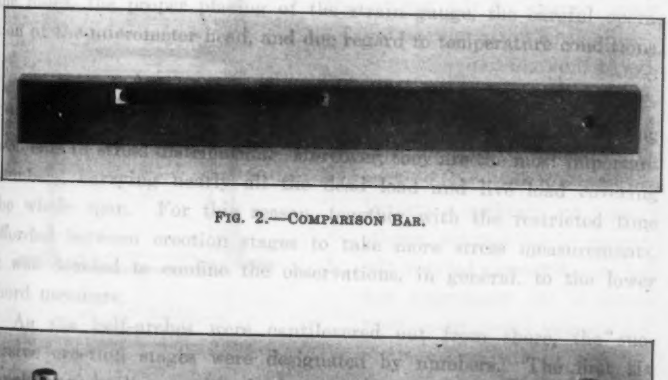


FIG. 2.—COMPARISON BAR.

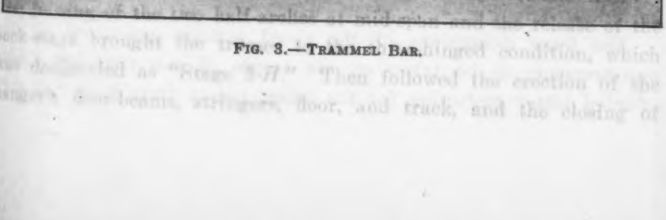


FIG. 3.—TRAMMEL BAR.

No expansion correction is necessary if the reference bar and the member have the same temperature; and, for this reason, the bar is allowed to rest on the member in order to equalize the temperature before the extensometer is applied. Nevertheless, two thermometers are provided, one permanently inserted in the face of the reference bar and the other to be placed between the gauge points on the member being measured. When the measurement is taken, the two thermometers are read [illegible] made for any difference between them.

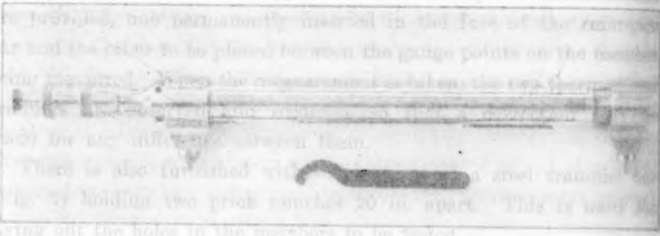


FIG. 1.—HOWARD EXTENSOMETER.

There is also furnished with the instrument a steel trammel bar (Fig. 3) holding two prick punches 20 in. apart. This is used for laying out the holes in the members to be tested.

The holes in the steel are drilled with a hand ratchet drill provided with combination bits, consisting of drill and countersink. The size of the drill is No. 57, and the countersink is 60 degrees. After drilling a hole, the edge between the countersink and cylindrical shaft is removed with a center punch which has been ground to an included angle of 45 degrees. With the tap of a hammer, the punch leaves a small conical seat to receive the measuring points of the extensometer, which [illegible] gauge [illegible] of advantages: The extreme tips of the measuring points are not used, thus eliminating the errors which would result from the unavoidable wear of these tips; the contact is on an area, instead of a point or line, thereby reducing the wear of the measuring points and the errors from compression of the bearing surfaces; the bearing area is depressed below the surface of the steel, so that it is better protected from surface dirt and mechanical injury. The countersunk holes aid in guiding the measuring points to their seat. [illegible] measurements.

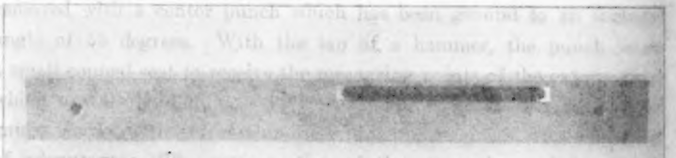


FIG. 2.—COMPARISON BAR.

Oil, vaseline, or ivory black paint are used for protecting, or covering the holes, depending on the interval between successive measurements. A pointed aluminum rod, a small cedarwood stick, and some absorbent cotton are used for cleaning the holes before taking any readings.

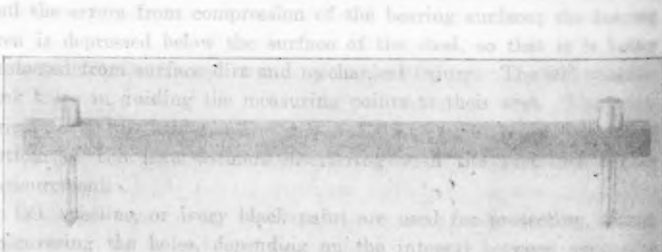


FIG. 3.—TRAMMEL BAR.

For the best operation of the instrument, two men are required: one to set the measuring points in the holes and hold the instrument square against the member; the other to operate the micrometer and take the readings.

The Howard strain gauge is probably the simplest and most accurate field instrument for measuring strains. It is free from the inertia and vibration errors which are inherent in automatic and recording types of extensometers; but it requires careful and experienced observers to attain the full degree of precision afforded by the instrument. Unless the readings are checked and all necessary precautions are observed, the apparent precision of the readings to 0.0001 in. will be illusory, and the measurements will be worthless.

Without mentioning all the precautions, most of which will readily suggest themselves to any one who has worked with instruments of precision, it may be stated in general that in order to get good results from the measurements, everything depends on the cleanliness of the holes, the proper placing of the strain gauge, the careful operation of the micrometer head, and due regard to temperature conditions.

Application to the Hell Gate Arch.

In the Hell Gate Arch, the lower chords present the most interesting problems in stress distribution. Moreover, they are the most important members, carrying nearly all the dead load and live load covering the whole span. For this reason, together with the restricted time afforded between erection stages to take more stress measurements, it was decided to confine the observations, in general, to the lower chord members.

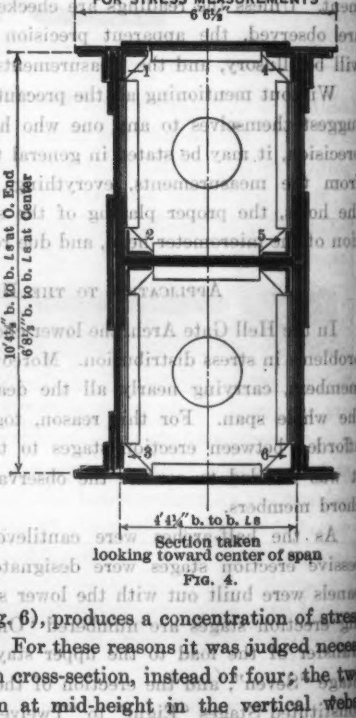
As the half-arches were cantilevered out from shore, the successive erection stages were designated by numbers. The first six panels were built out with the lower stay acting, and the corresponding erection stages are numbered "One" to "Six", respectively. The transfer of the load to the upper stay produced the condition called Stage "Seven", and the erection of the seventh to the eleventh panels constituted Stages "Eight" to "Twelve", respectively. (Plate XLIV.) The joining of the two half-arches at mid-span and the release of the back-stays brought the trusses to the three-hinged condition, which was designated as "Stage 3-*H*." Then followed the erection of the hangers, floor-beams, stringers, floor, and track, and the closing of

the crown-hinge, bringing the arch to the final or two-hinged condition known as "Stage 2-*H*."

To secure full information as to the distribution of stress in any member, it is necessary to take measurements at not less than three, and preferably four, points of the cross-section near each end of the member. The intensity of stress at any other point of the member can then be found by planar and linear interpolation.

The lower chords of the Hell Gate Arch have a double rectangular section (Fig. 4), consisting of two compartments separated by a horizontal diaphragm. On account of the mass of metal concentrated at and near this diaphragm, measurements at the four extreme corners of the section would not be fairly representative of the conditions throughout the entire area. Any difference of temperature between the horizontal diaphragm and the outside webs tends to produce internal stresses in the member, resulting in a difference between the stress in the diaphragm and the average stress throughout the rest of the cross-section. Furthermore, the beveling of the outer thirds of the webs (Fig. 6), produces a concentration of stress in the middle third of the joint. For these reasons it was judged necessary to take six readings at each cross-section, instead of four; the two additional readings being taken at mid-height in the vertical webs, as indicated in Fig. 4.

SECTION OF BOTTOM CHORD OF HELL GATE ARCH BRIDGE SHOWING LOCATION OF GAUGE POINTS FOR STRESS MEASUREMENTS



Section taken looking toward center of span

FIG. 4.

The large dimensions of the chord sections permitted all the measurements to be taken on the inside of the members. This afforded better protection for the gauge points from rain and dirt

PLATE XLIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1418.
STEINMAN ON
STRESS MEASUREMENTS,
HELL GATE ARCH BRIDGE.

MEMBER		0-2				2-4				4-6			
		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary	
Erection Stage	TRUSS	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum
1	S. Truss, Long Is. Side	230	400	1260	122								
	N. Truss, Long Is. Side	—	—										
	N. Truss, Wards Is. Side	—	—										
2	S. Truss, Long Is. Side	695	925	320	24	—	—	—	—				
	N. Truss, Long Is. Side	695	1125	320	90	—	—	—	—				
	N. Truss, Wards Is. Side	600	925	320	93	447	700	835	70				
3	S. Truss, Long Is. Side	—	—	—	—	828	610	225	181	—	—	—	—
	N. Truss, Long Is. Side	—	—	—	—	828	860	225	214	—	—	—	—
	N. Truss, Wards Is. Side	1660	1010	102	7	828	890	225	56	480	440	385	77
4	S. Truss, Long Is. Side	2010	1890	58	10	1475	1590	85	61	800	890	101	97
	N. Truss, Long Is. Side	2010	1640	58	134	1175	1475	85	66	800	1150	101	65
	N. Truss, Wards Is. Side	2010	1850	58	10	[illegible]	2010	85	48	1070	950	101	71
5	S. Truss, Long Is. Side	3190	2925	23	26	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Wards Is. Side	2640	2425	28	8	2230	2140	58	58	1440	1460	76	75
6	S. Truss, Long Is. Side	—	—	—	—	3520	3165	30	11	2630	2550	51	28
	N. Truss, Long Is. Side	3660	3110	6	68	3520	3300	30	34	2410	2600	51	54
	N. Truss, Wards Is. Side	3580	3260	6	12	3037	3040	30	60	2354	2160	51	50
7	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	2600	—	—	—	1480	—	—	—	725	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
8	S. Truss, Long Is. Side	2570	1525	96	37	2000	1890	104	58	1310	1175	103	96
	N. Truss, Long Is. Side	2570	1540	95	81	2000	1910	104	110	1310	1075	102	45
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
9	S. Truss, Long Is. Side	—	—	—	—	2570	2375	74	51	—	—	—	—
	N. Truss, Long Is. Side	3040	1775	72	45	—	—	—	—	1920	1425	58	47
	N. Truss, Wards Is. Side	3110	2075	74	30	—	—	—	—	—	—	—	—
10	S. Truss, Long Is. Side	3640	2825	53	8	—	—	—	—	2830	2275	38	30
	N. Truss, Long Is. Side	—	—	—	—	3280	2840	53	63	—	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
11	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	4800	—	—	—	4100	—	—	—	3720	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
3 H	S. Truss, Long Is. Side	7200	5910	12	—	7130	6850	9	—	6330	5775	9	—
	N. Truss, Long Is. Side	6340	5075	12	—	6060	5310	9	—	5620	5150	9	—
	N. Truss, Wards Is. Side	5600	5025	12	—	5800	5845	9	—	5300	5190	9	—
2 H	S. Truss, Long Is. Side	—	—	—	—					—	—	—	—
	N. Truss, Long Is. Side	11890	10150	2	23					11500	10990	5	13
	N. Truss, Wards Is. Side	11890	11450	2	3					11500	11650	5	11

MEMBER		6-8				8-10				10-12			
		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary	
Erection Stage	TRUSS	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum
1	S. Truss, Long Is. Side												
	N. Truss, Long Is. Side												
	N. Truss, Wards Is. Side												
2	S. Truss, Long Is. Side												
	N. Truss, Long Is. Side												
	N. Truss, Wards Is. Side												
3	S. Truss, Long Is. Side												
	N. Truss, Long Is. Side												
	N. Truss, Wards Is. Side												
4	S. Truss, Long Is. Side	232	475	235	118								
	N. Truss, Long Is. Side	—	—	—	—								
	N. Truss, Wards Is. Side	—	—	—	—								
5	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—				
	N. Truss, Long Is. Side	—	—	—	—	—	—	—	—				
	N. Truss, Wards Is. Side	585	425	140	115	230	250	—	—				
6	S. Truss, Long Is. Side	1600	2210	105	44	940	1290	161	195	282	725	479	236
	N. Truss, Long Is. Side	1600	1375	105	67	900	1040	161	108	282	625	479	72
	N. Truss, Wards Is. Side	1379	1275	105	80	720	690	161	147	—	—	—	—
7	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	587	—	—	—	150	—	—	—	282	850	33	6
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
8	S. Truss, Long Is. Side	830	1090	235	86	815	1050	300	131	940	1040	108	130
	N. Truss, Long Is. Side	830	410	235	136	816	1000	300	98	918	980	108	88
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
9	S. Truss, Long Is. Side	—	—	—	—	1700	1550	132	78	—	—	—	—
	N. Truss, Long Is. Side	1560	1100	113	17	—	—	—	—	1880	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
10	S. Truss, Long Is. Side	2410	2450	65	26	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	—	—	—	—	2760	2880	195	70	3080	—	106	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
11	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	4470	3990	97	33
	N. Truss, Long Is. Side	3390	—	—	—	4000	—	—	—	—	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
3 H	S. Truss, Long Is. Side	5620	5610	11	—	5770	5720	12	—	5500	5640	18	—
	N. Truss, Long Is. Side	4730	4910	11	—	5180	5510	12	—	5150	5040	18	—
	N. Truss, Wards Is. Side	4930	4310	11	—	4920	4775	12	—	5080	5040	18	—
2 H	S. Truss, Long Is. Side					—	—	—	—				
	N. Truss, Long Is. Side					11890	11225	4	13				
	N. Truss, Wards Is. Side					11290	10675	4	7				

MEMBER		12-14				14-16				16-18			
		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary	
Erection Stage	TRUSS	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum
8	S. Truss, Long Is. Side	310	690	530	102								
	N. Truss, Long Is. Side	310	690	530	158								
	N. Truss, Wards Is. Side												
9	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	1930	—	—	—	339	—	—	—	—	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
10	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	2080	—	77	—	1120	—	83	—	375	—	165	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
11	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	3340	—	—	—	2240	—	—	—	1850	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
3 H	S. Truss, Long Is. Side	5300	5360	21	—	5600	5770	26	—	—	—	—	—
	N. Truss, Long Is. Side	5300	6050	21	—	5580	5990	28	—	6000	6500	26	—
	N. Truss, Wards Is. Side	5330	6090	24	—	5550	6080	28	—	6030	6080	32	—
2 H	S. Truss, Long Is. Side	—	—	—	—					—	—	—	—
	N. Truss, Long Is. Side	11260	11610	6	10					11610	11720	7	16
	N. Truss, Wards Is. Side	11260	11015	6	5					11610	11310	7	7

MEMBER		18-20				20-22				22-22A			
		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary	
Erection Stage	TRUSS	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum
8	S. Truss, Long Is. Side												
	N. Truss, Long Is. Side												
	N. Truss, Wards Is. Side												
9	S. Truss, Long Is. Side												
	N. Truss, Long Is. Side												
	N. Truss, Wards Is. Side												
10	S. Truss, Long Is. Side												
	N. Truss, Long Is. Side												
	N. Truss, Wards Is. Side												
11	S. Truss, Long Is. Side	—	—	—	—								
	N. Truss, Long Is. Side	495	—	—	—								
	N. Truss, Wards Is. Side	—	—	—	—								
3 H	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	6470	6810	37	—	6750	7340	38	—	7150	7070	45	—
	N. Truss, Wards Is. Side	6480	6700	34	—	6750	7020	48	—	—	—	—	—
2 H	S. Truss, Long Is. Side					—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side					11980	12175	11	16	11650	11775	8	27
	N. Truss, Wards Is. Side					—	—	—	—	—	—	—	—

NOTES

All Primary Stresses are compression, and are given in pounds per square inch.

All Secondary Stresses are either tension or compression, representing equal values of opposite sign in the top and bottom fibers of a section.

All Secondary Stresses are given in percentages of the corresponding Primary Stresses.

Each Calculated Primary Stress is calculated for the actual weight and condition of structure and travelers at the time the corresponding stress was measured.

Each Measured Primary Stress is the average of twelve strain measurements, six points being observed at each end of each member.

Each Calculated Secondary Stress is the larger of the two secondary stresses calculated for the two ends of each member in each erection stage.

Each Measured Secondary Stress is the larger of the two secondary stresses found for the two ends of each member in each erection stage, the secondary stress at each end being obtained as one-half the difference between the average stresses in the top and bottom fibers of the cross-section.

For Stage 3-H, the Calculated Secondary Stresses were figured for the theoretical stage with no floor-beams or hangers erected.

As the field measurements were taken during the erection of floor-beams and hangers, with varying positions of the erection travelers, the Measured Secondary Stresses for this stage are not properly comparable with the Calculated Secondary Stresses and are therefore omitted from the tabulation.

EXTENSOMETER MEASUREMENTS OF STRESSES IN BOTTOM-CHORD MEMBERS OF HELL GATE ARCH BRIDGE.

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Incidentally, it secured better protection for the observers from falling drift-pins and rivets.

The gauge points were drilled and the initial readings taken while the members were still on the dock or on the car-floats. The points were located in the vertical legs of the 8 by 8-in. flange angles connecting the vertical and horizontal webs, and as near the panel points as possible, but not within gusset or splice-plates. There were thus twelve gauge points in each member, each gauge point consisting of two holes 20 in. apart. The exact location of each gauge point was noted for permanent record.

For designation, the gauge points were numbered from 1 to 6 at each section, and to these figures was prefixed the number of the member. Thus, "hole 681" denotes the upper, left-hand gauge point near panel point 6 of member 6-8; similarly, "hole 864" denotes the upper right-hand gauge point (No. 4) near panel point 8 of the same member.

The number of the hole and that of the erection stage thus suffice to identify any extensometer reading.

Because of the more rapid erection on the Wards Island side, one half-truss on that side had to be omitted from the programme of measurements. The remaining three sets of measurements, however, are sufficient to furnish information as to any variations from the symmetrical disposition of stresses about the longitudinal and transverse center lines of the bridge.

There were thus twelve pairs of holes in each of thirty-five chord members to be drilled; and stress readings were to be taken at these points in fourteen different stages. This meant a total of more than 3 000 extensometer measurements in the regular schedule, besides special measurements in the back-stays and the eye-bar connection links.

In some of the later stages, time did not permit all the chord stresses to be measured; and, therefore, alternate members were omitted.

On account of the difficulty of traveling through the members with the instruments, involving crawling through numerous diaphragm holes, much time was consumed in going to and from the points of measurement. Consequently, it was seldom possible to measure more than two members in a day.

TABLE 1.—Typical Field Record Sheet.

Record of Extensometer Readings at Hell Gate Arch Bridge.

Location: Long Island End, Member 0-2, South Truss, and Member 2-4, North Truss.

Date: Sept. 15, 1915. Time: 8.30 A. M. to 4.15 P. M. Weather: Clear.

Observers: O. H. S. Koch and A. V. O'Donnell.

No.	Temperature, Degrees Fahr.			Reading.		Difference in Length between Bar and Member.		Length corrected to 60° Fahr.	Location on member where measurements were taken.
	Air.	Bar.	Member.	Bar.	Member.	Measured.	Corrected for temp.		
				South Truss, Member 0-2.					
2025	......	78.5	78	9983	9877	−0106	−0105	19.9895	025
2026	......	80	78	9978	9850	−0128	−0125	19.9875	024
2027	......	80	78.5	9977	9855	−0122	−0120	19.9880	021
2028	......	80	79.5	9977	0139	+0162	+0163	20.0163	022
2029	......	81	78	9978	9898	−0080	−0076	19.9924	205
2030	......	81	79	9978	9935	−0043	−0040	19.9960	204
2031	......	82	80	9978	9739	−0239	−0236	19.9764	201
2032	......	82	80	9978	9878	−0100	−0097	19.9903	202
2033	......	83	81	9981	9896	−0085	−0082	19.9918	023
2034	......	83	79.5	9980	9966	−0014	−0009	19.9991	026
2035	......	83	80	9980	9892	−0088	−0084	19.9916	206
2036	84	83.5	84	9980	9930	−0050	−0051	19.9949	203
				North Truss, Member 2-4.					
2037	......	85.5	83	9982	9952	−0030	−0027	19.9973	245
2038	......	85.5	87	9982	9975	−0007	−0009	19.9991	244
2039	......	86	86	9982	9995	+0013	+0013	20.0013	242
2040	......	86	88	9981	9950	−0031	−0034	19.9966	241
2041	......	86.5	85.5	9982	9990	+0008	+0009	20.0009	422
2042	......	86.5	89	9981	9919	−0062	−0065	19.9935	421
2043	......	87	83.5	9981	0013	+0032	+0037	20.0037	425
2044	......	87	87.5	9981	0340	+0359	+0358	20.0358	424
2045	......	87	86	9983	9991	+0008	+0009	20.0009	246
2046	......	87	90.5	9982	0045	+0063	+0058	20.0058	243
2047	......	87.5	87	9981	9974	−0007	−0006	19.9994	423
2048	86	87.5	86	9981	0008	+0027	+0029	20.0029	426

Notes.—No. = serial number of observation. Under temperature: air = atmospheric temp., bar = temp. of std. reference bar, member = temp. of member or metal. Under reading: bar = strain gauge measurement of std. reference bar, member = strain gauge measurement of member or metal.

Remarks.—Erection Stage No. 10, traveler still at 15–17. Wind: southwest, mild.

Signed, Otto H. S. Koch.

Table 1 shows a typical field-office record sheet. It also serves to indicate the arrangement in the field notebook, as the record sheet is practically a typewritten copy of the field entries.

From the record sheets the measurements were entered and computed on office cards, illustrated by Tables 2, 3, and 4. One of these cards was provided for each member, with spaces for entering the

observed strains in each erection stage. Following the numbers or marks of the gauge points there is a column headed "Initial" in which are recorded the lengths between gauge points before the member is stressed. The standard length of 20 in. is understood to be added to each of these figures. In the remaining columns the figures denote the actual strain, that is the increase or decrease from the initial gauge length for the different erection stages. Each unit of strain (0.0001 in.) represents a fiber stress of 150 lb. per sq. in.; hence the sum of the six strains at a section, multiplied by 25, gives the average unit stress in the member. The calculated stresses, inserted for comparison with the measured stresses, were computed from the shipping weights of the erected members, with an allowance for erection material.

The extreme strain readings observed for each member, multiplied by 150, give the minimum and maximum fiber stresses in the member.

From the cards the summary table (Plate XLIV) was compiled, giving a comparison of the calculated and the observed stresses in the various erection stages.

The secondary stresses were computed from the strain measurements, as follows: The average of the observed strains in Holes 1 and 4 at any section gave the upper fiber stress; the average for Holes 3 and 6 gave the bottom fiber stress; one-half the difference between these two extreme fiber stresses gave the secondary stress. This result divided by the average stress in the member gave the secondary stress as a percentage of the primary stress.

Results of the Stress Measurements.

A tabulation of the results of the stress measurements on the Hell Gate Arch during erection is shown on Plate XLIV. The notes and data on that plate render it self-explanatory.

The results of the observations have been studied from various angles, in order to determine the following relations:

1.—Comparison of the measured average stresses at the two ends of each member, as an index of the precision of the method;

2.—Comparison of measured with calculated primary stresses;

3.—Comparison of extreme secondary stresses with primary stresses, in order to establish empirical relations between the two;

TABLE 2.—TYPICAL OFFICE RECORD CARD.

EXTENSOMETER MEASUREMENTS OF ERECTION STRESSES, HELL GATE ARCH.

Wards Island End. North Truss. Member 0-2.

	Hole.	Initial.	Stage 1.	Stage 2.	Stage 3.	Stage 4.	Stage 5.	Stage 6.	Stage 7.	Stage 8.	Stage 9.	Stage 10.	Stage 11.	3-*H*.	2-*H*.
Lower end.	021	— 0020	....	— 10	— 12	— 16	— 23	— 26	....	....	— 24	....	....	— 39	— 77
	022	+ 0018	....	0	— 7	— 13	— 15	— 17	....	....	— 16	....	....	— 34	— 83
	023	— 0007	....	+ 1	— 12	— 13	— 20	— 27	....	....	— 13	....	....	— 36	— 73
	024	+ 0011	....	— 13	— 5	— 12	— 16	— 19	....	....	— 23	....	....	— 38	— 60
	025	+ 0048	....	— 12	— 3	— 15	— 14	— 16	....	....	— 19	....	....	— 38	— 76
	026	— 0031	....	— 1	— 4	— 12	— 14	— 24	....	....	— 17	....	....	— 33	— 67
Total.....		....	....	— 35	— 43	— 81	— 102	— 129	....	....	— 112	....	....	— 218	— 436
Aver. Stress..		....	....	— 875	— 1 075	— 2 025	— 2 550	— 3 225	....	....	— 2 800	....	....	— 5 450	— 10 900
Upper end.	201	+ 0002	....	— 3	— 7	— 9	— 18	— 15	....	....	— 13	....	....	— 29	— 61
	202	— 0001	....	— 11	— 13	— 13	— 22	— 25	....	....	— 17	....	....	— 35	— 96
	203	+ 0002	....	+ 2	— 7	— 11	— 17	— 24	....	....	— 8	....	....	— 34	— 75
	204	+ 0008	....	— 18	— 6	— 16	— 14	— 23	....	....	— 31	....	....	— 49	— 72
	205	+ 0025	....	— 5	— 1	— 10	— 10	— 21	....	....	— 18	....	....	— 40	— 108
	206	+ 0011	....	— 4	— 6	— 9	— 11	— 24	....	....	— 15	....	....	— 37	— 68
Total.....		....	....	— 39	— 40	— 68	— 92	— 132	....	....	— 102	....	....	— 224	— 480
Aver. stress..		....	....	— 975	— 1 000	— 1 700	— 2 300	— 3 300	....	....	— 2 550	....	....	— 5 600	— 12 000
Calc. stress..		....	....	— 609	— 1 660	— 2 010	— 2 660	— 3 580	....	....	— 3 110	....	....	— 5 900	— 11 890
Min. stress..		....	....	+ 300	— 150	— 1 350	— 1 500	— 2 250	....	....	— 1 200	....	....	— 4 350	— 9 000
Max. stress..		....	....	— 2 700	— 1 950	— 2 400	— 3 450	— 4 050	....	....	— 4 650	....	....	— 7 350	— 16 200

TABLE 3.—TYPICAL OFFICE RECORD CARD.

EXTENSOMETER MEASUREMENTS OF ERECTION STRESSES, HELL GATE ARCH.

Long Island End. North Truss. Member 2-4.

	Hole.	Initial.	Stage 1.	Stage 2.	Stage 3.	Stage 4.	Stage 5.	Stage 6.	Stage 7.	Stage 8.	Stage 9.	Stage 10.	Stage 11.	3-H.	2-H.
Lower end.	241	— 0017	..	..	— 3	— 5	..	— 9	..	— 8	..	— 17	..	— 32	....
	242	+ 0024	..	..	— 4	— 2	..	— 16	..	— 5	..	— 11	..	— 31	....
	243	+ 0075	..	..	— 2	— 6	..	— 27	..	— 10	..	— 17	..	— 34	....
	244	+ 0015	..	..	— 12	— 12	..	— 22	..	— 22	..	— 24	..	— 38	....
	245	— 0012	..	..	— 9	— 14	..	— 22	..	— 11	..	— 15	..	— 36	....
	246	+ 0032	..	..	— 10	— 25	..	— 34	..	— 16	..	— 23	..	— 47	....
Total......		....	..	..	— 40	— 64	..	— 180	..	— 72	..	— 107	..	— 218	....
Aver. stress...		....	..	..	— 1 000	— 1 600	..	— 3 250	..	— 1 800	..	— 2 675	..	— 5 450	....
Upper end.	421	— 0036	..	..	— 18	— 11	..	— 28	..	— 28	..	— 29	..	— 44	....
	422	+ 0023	..	..	+ 1	— 4	..	— 18	..	— 7	..	— 14	..	— 33	....
	423	— 0001	..	..	+ 12	— 1	..	— 16	..	— 2	..	— 5	..	— 28	....
	424	+ 0386	..	..	— 16	— 18	..	— 29	..	— 31	..	— 28	..	— 45	....
	425	+ 0057	..	..	— 12	— 18	..	— 23	..	— 12	..	— 20	..	— 42	....
	426	+ 0037	..	..	+ 4	— 2	..	— 20	..	— 1	..	— 8	..	— 33	....
Total......		....	..	..	— 29	— 54	..	— 134	..	— 81	..	— 104	..	— 225	....
Aver. stress...		....	..	..	— 725	—1 350	..	— 3 350	..	— 2 025	..	— 2 600	..	— 5 625	....
Calc. stress...		....	..	..	— 828	—1 475	..	— 3 520	..	— 2 000	..	— 3 280	..	— 6 050	....
Min. stress...		....	..	..	+1 800	— 150	..	— 1 350	..	— 150	..	— 750	..	— 4 200	....
Max. stress...		....	..	..	—2 700	—3 750	..	— 5 100	..	— 4 650	..	— 4 350	..	— 7 050	....

4.—Comparison of calculated with measured secondary stresses, in order to ascertain the effects of special features of fabrication and erection on these stresses; and

5.—Comparison of calculated with measured extreme fiber stresses, in order to determine the resultant effect of all variations in primary and secondary stress.

COMPARISON OF MEASUREMENTS AT THE TWO ENDS OF EACH MEMBER.

By averaging the six measurements near each end of a member, two values are obtained for the average intensity of stress in the member. The difference between these two values, provided there is

TABLE 4.—Typical Office Record Card.

Extensometer Measurements of Erection Stresses, Hell Gate Arch.

Wards Island End. North Truss. Member 4-6.

Hole.		Initial.	Stage 1.	Stage 2.	Stage 3.	Stage 4.	Stage 5.	Stage 6.	Stage 7.	Stage 8.	Stage 9.	Stage 10.	Stage 11.	3-*H*.	2-*H*.
Lower end.	461	— 0078	..	..	— 12	— 11	— 17	— 15	..	..	..	..	..	— 37	— 62
	462	+ 0015	..	..	— 7	— 6	— 17	— 13	..	..	..	..	..	— 35	— 75
	463	— 0037	..	..	— 14	— 12	— 14	— 27	..	..	..	..	..	— 48	— 76
	464	— 0016	..	..	+ 9	+ 4	+ 5	— 1	..	..	..	..	..	— 20	— 51
	465	— 0024	..	..	+ 4	— 5	— 11	— 14	..	..	..	..	..	— 35	— 76
	466	— 0068	..	..	+ 2	— 9	— 5	— 18	..	..	..	..	..	— 38	— 70
Total.....		....	..	..	— 18	— 39	— 59	— 88	..	..	..	..	..	— 208	— 410
Aver. stress..		....	..	..	— 450	— 975	—1 475	—2 200	..	..	..	..	..	— 5 200	— 10 250
Upper end.	641	— 0032	..	..	— 4	— 14	— 21	— 17	..	..	..	..	..	— 44	— 70
	642	— 0012	..	..	— 5	— 8	— 14	— 17	..	..	..	..	..	— 42	— 90
	643	+ 0023	..	..	— 5	— 3	— 1	— 15	..	..	..	..	..	— 32	— 69
	644	— 0004	..	..	— 2	— 4	— 10	— 8	..	..	..	..	..	— 28	— 81
	645	— 0367	..	..	— 1	— 11	— 11	— 15	..	..	..	..	..	— 34	— 139
	646	+ 0040	..	..	0	+ 3	— 1	— 13	..	..	..	..	..	— 27	— 73
Total.....		....	..	..	— 17	— 37	— 58	— 85	..	..	..	..	..	— 207	— 522
Aver. stress..		....	..	..	— 425	— 925	—1 450	—2 125	..	..	..	..	..	—5 175	— 13 050
Calc. stress..		....	..	..	— 480	—1 070	—1 440	—2 354	..	..	..	..	..	—5 300	— 11 500
Min. stress...		....	..	..	+1 350	+ 600	+ 750	— 150	..	..	..	..	..	—3 000	— 7 650
Max. stress...		....	..	..	—2 100	—2 100	—3 150	—4 050	..	..	..	..	..	—6 600	— 20 850

no inequality of respective cross-sections, is an index of the precision of the observations. The average value of this difference for all the lower chord measurements during erection was 140, and the greatest value was 500 lb. per sq. in. Consequently, by the theory of errors, the result obtained by averaging the two end stresses has an average probable error of 47 and a maximum probable error of 170 lb. per sq. in.

The foregoing figures represent the combined effect of all personal and instrumental errors in taking the observations. A very small error in each of the instrumental readings will account for these results. Thus, an inaccuracy of 0.0002 in. in each micrometer reading,

and of 1° Fahr. in each temperature reading, will produce the foregoing average probable error, even if all inaccuracies compensate according to the theory of probabilities. A very small fraction of the same inaccuracies will suffice to account for the observed discrepancies, if the inaccuracies do not compensate fully according to the probability theory. As the least reading of the micrometer is 0.0001 and of the thermometer is 1° Fahr., the results given indicate that the work, as a whole, was carefully executed, and that the utmost precision afforded by the method was actually attained.

Another fact to be observed is that the magnitude of the differences between the end measurements has no connection with the magnitude of the respective stresses. This method of measuring the stresses makes the errors independent of the intensity of the stress. No matter how small the stress, the result, as just shown, may be in error by as much as 170 lb. per sq. in. These considerations indicate the unsuitableness of this method of stress measurement for the smaller stresses, as the results would be too erratic. Although the stresses measured on the Hell Gate Bridge ranged from 400 to 20 000, the writer would not recommend the use of the extensometer for stresses of less than 1 000 lb. per sq. in.

The foregoing discussion of results does not include the final stage (2-*H*). In that stage, in the members near the ends of the span, the upper end in every case was found to present a larger average stress than did the lower end. The differences ranged from 450 to 2 800 lb. per sq. in. This anomalous result appears to arise from some unexplained disturbance of stress distribution, and does not represent any error of observation.

COMPARISON OF MEASURED WITH CALCULATED STRESSES.

A comparison of measured with calculated stresses discloses a greater discrepancy than is found between the measurements at the two ends of a member. This is due to other factors, besides instrumental and personal inaccuracies, entering into consideration.

The results of the comparison, including all measurements on chords, eye-bars, and back-stays, are as follows:

For stresses in the ranges 0-1 000, 1 000-3 000, 3 000-5 000, 5 000-10 000, and 10 000-20 000, the average percentage discrepancies between the calculated and measured stresses were 50, 17, 13, 8, and 6%, respec-

tively. These percentages indicate that the method is of little value for checking calculated stresses of less than 1 000 lb. per sq. in., but is of increasing value as the intensity of stress increases.

The average discrepancies in the foregoing ranges were 223, 313, 498, 525, and 829 lb. per sq. in., respectively, and may be expressed very closely by the formula,

$$\left.\begin{array}{l}\text{Average discrepancy between}\\ \text{calculated and measured stress}\end{array}\right\} = 5\% \text{ of the stress} + 200.$$

The greatest discrepancies in the foregoing ranges were 535, 1 045, 1 285, 1 290, and 1 740, respectively, and may be expressed approximately by the formula,

$$\left.\begin{array}{l}\text{Maximum discrepancy between}\\ \text{calculated and measured stress}\end{array}\right\} = 5\% \text{ of the stress} + 1\,000.$$

It will be observed that these formulas contain both a percentage factor and an absolute term, indicating that the discrepancy between calculation and measurement is the resultant of two groups of errors, one dependent on, and the other independent of, the magnitude of the stress.

The absolute term in these formulas (average 200, maximum 1 000 lb. per sq. in.) may be assumed to represent the actual inaccuracy of the measurements, or the total effect of personal, instrumental, and physical errors.

The most important of the physical errors are those due to thermal effects. An error of 1° Fahr. in determining the temperature of the steel at the point of measurement, or a change of 1° Fahr. in the temperature of the instrument during any observation, produces an error of 195 lb. per sq. in. in the resulting stress. The first error named is not necessarily due to incorrect reading of the thermometer, but may be caused by a difference in temperature between the inside and the outside of the thick webs composing the members. Such errors, as a rule, are not compensating; they would naturally occur in the same direction at both ends of a member, and, therefore, would not appear in the difference between the two end measurements.

Other errors arise from differences in temperature between different points of the large chord sections. Thus, on a sunny day, the outside webs and covers may be hotter than the middle diaphragm. The resulting differences of expansion produce a redistribution of

the stress in the cross-section. Although the actual average stress throughout the entire section must remain unchanged, the measured average stress will be affected by these internal temperature strains, as the readings are taken at a limited number of points in the cross-section.

It is difficult to estimate the total probable error due to these effects of temperature. Nevertheless, from a study of the results of the measurements, it appears that the effect of the combined temperature errors, together with any other unascertainable physical effects, is to increase the probable error of the stresses, as previously determined, from 47 (average) and 170 (maximum) to 200 (average) and 1 000 (maximum).

The foregoing discussion should serve to emphasize the importance of careful attention to temperature conditions in taking observations; and it explains why cloudy days should be selected in order to obtain the most accurate results.

After deducting all personal, instrumental, and temperature errors, represented by the absolute term in these formulas, there still remains a discrepancy between the calculated and the measured stresses represented by the percentage term (5%). This covers all percentage errors in the measured and the calculated stresses, that is, all errors which are proportional to the magnitude of the stress.

In the measured stresses, the only percentage error arises from variations in the value of the modulus of elasticity from the assumed value of 30 000 000. Such variation does not affect the accuracy of the measurements, but simply the conversion of the measured strains into intensities of stress. The resulting stress is then affected by a percentage error equal to the percentage variation in the value of E, and this may amount to $\pm$ 3 per cent.

In the calculated stresses, all errors are percentage errors. They are caused principally by variations in the loading and cross-sections from the values assumed. In the assumed loads there is a probable error of about $\pm$ 3%, on account of the uncertainty in allowing for the weight of traveler, track, rivets, pins, staging, etc. In the cross-sections there is a probable error of $\pm$ 2% on account of the difference between actual and theoretical sections.

The combined effect of the foregoing errors in assuming the loading, cross-sections, and value of E, will account for the percentage

term ($\pm$ 5%) in the discrepancy between calculated and measured stresses.

The major part of this discrepancy of 5% arises from the errors in the calculated stress. This demonstrates the futility of excessive refinement in the calculation of stresses.

COMPARISON OF EXTREME SECONDARY STRESSES WITH PRIMARY STRESSES.

The object of this comparison was to establish a relation between the average stresses and the extreme variations from these average stresses in the members of the structure. The information derived from such comparisons should be of value in guiding the selection and specification of working stresses for bridge materials. For this purpose it appeared desirable to get the extreme variation of fiber stress in each member, without regard to the cause or causes producing it. Such secondary stresses would include, not only the stress from bending of the members in the plane of the truss, as usually computed, but also any horizontal or lateral bending stresses from wind and transverse strains, effect of shop inaccuracies, internal temperature strains, lack of uniformity of material, and any other possible causes.

As a fair measure of this extreme secondary stress, including the resultant effect of all possible contributing elements, the difference was taken between the average of the twelve measurements in a member and that one of the twelve measurements departing most widely from the average.

A comparison of the secondary stresses thus obtained with the corresponding primary (or average) stresses yielded the following results:

For primary stresses in the ranges 0-1 000, 1 000-2 000, 2 000-3 000, 3 000-5 000, 5 000-7 000, and 7 000-8 000, the percentages of extreme secondary stress averaged 268, 148, 71, 59, 37, and 32, respectively. This illustrates the diminishing relative importance of secondary stresses with increasing primary stress.

Although the percentages diminish in the foregoing series of ranges, the absolute amounts of the secondary stress show a small increase, averaging 1 580 lb. per sq. in. in the lowest range and 2 370 lb. per sq. in. in the highest range.

In the one hundred cases represented in the foregoing results, the greatest extreme secondary stress that appears is 3 700 lb. per sq. in.

Plotting the results with primary and extreme secondary stresses, respectively, as co-ordinates, nearly all the observations were found to be included in a belt between two lines having a 12% slope. From this may be deduced the relation,

$$\left.\begin{array}{l}\text{Extreme secondary}\\ \text{stress in a member}\end{array}\right\} = 12\% \text{ of primary stress} + K,$$

where K varies from 600 to 2 600, with an average value of 1 600 lb. per sq. in.

The form of this expression, a percentage term plus an absolute term, indicates that the measured secondary stresses include effects proportional to the direct stress in the member as well as effects independent of that stress. The latter effects, constituting the major portion of the measured secondary stresses, are produced by such causes as the bending of a member due to its own weight, wind and lateral strains, internal temperature strains, erection strains, etc.

The proportional component or percentage term in the formula represents the secondary stresses resulting from the direct primary strains. It amounts to only 12% of the primary stresses in the case of the Hell Gate chords. It is interesting to note that this component of the secondary stress is smaller than the combined effect of the other contributions which are generally omitted from consideration in the computation of secondary stresses.

Combining the results of this and the preceding comparison, we may establish certain conclusions for guidance in specifying extreme working stresses for bridge members. As the maximum discrepancy between calculated and measured stress is 5% + 1 000, and as a possible extreme value of the secondary stress is 12% + 2 600, it appears that the sum of these two variations should be left as a necessary margin between the elastic limit of the material and the maximum calculated primary stress. As the Hell Gate chords had certain special features tending to reduce the secondary stresses, it is probable that the 12% factor is not typical, and that about 20% would be a better allowance for most bridges. Hence, about 25% + 4 000 should be deducted from the minimum elastic limit of the material in order to obtain the safe working stress for bridge members.

It is recognized, of course, that a rule of this form can properly apply only to structures in which the secondary stresses have normal

or average values. Where unusually large secondaries may be anticipated, these should receive special investigation and attention.

COMPARISON BETWEEN CALCULATED AND MEASURED SECONDARY STRESSES.

In order to be comparable with the calculated secondary stresses due to bending of the members in the plane of the truss, the "measured secondary stresses" had to be defined as one-half the difference between the average measured stresses in the top and bottom fibers of the cross-section. Any lateral variation in the stress had to be ignored.

The "calculated secondary stresses" were obtained by analytical methods for Stages 2, 4, 6, 8, 10, 3-*H*, and 2-*H*, and by interpolation for the intermediate stages.

Both the measured and the calculated values were reduced to percentages of the corresponding primary stresses and recorded in the table, Plate XLIV.

It should be noted that the high percentages always occur with low primary stresses. The law of variation is asymptotic.

The average of the calculated secondary stresses, for all bottom chord members and all erection stages, was about 1 050 lb. per sq. in. The average of all measured secondary stresses was only 700 lb. per sq. in.

The highest calculated secondary stress in any stage was 2 920; and the corresponding measured stress was 1 990 lb. per sq. in.

A comparison of individual values of calculated and of measured secondary stresses is not very illuminating, as any systematic variations are more or less obscured by the effects of erratic readings and other disturbing factors.

More instructive results are obtained by averaging the individual readings in groups so as to eliminate accidental variations and disclose the true relations between the calculated and the measured values.

If the secondary stresses are grouped and averaged in ranges of percentages, the following comparison is obtained:

In the ranges,

0-20%, 20-40%, 40-100%, 100-300%, and more than 300%,

the average calculated secondary stresses were:

6%, 30%, 69%, 143%, and 473%,

and the corresponding average measured secondary stresses were:

26%, 28%, 46%, 93%, and 110%, respectively.

From these figures, the following relations are evident:

1.—Except for the smaller percentage of secondary stresses, the measured values are consistently lower than the calculated values.

2.—As the percentage of calculated stresses increases, the percentage of measured values also increases, but not as rapidly.

3.—For the highest percentages of secondary stresses, the measured values amount to only a small fraction of the calculated values.

The lower values of the measured secondary stresses are partly due, in the case of the Hell Gate Arch, to the three-faced joints in the bottom chords; but, to a large extent, the effect is to be ascribed to a readjustment of strains tending to relieve the higher secondary stresses. As this effect obtains in all structures, it may be concluded that the actual secondary stresses are generally lower than the calculated values.

These results may be expressed by an approximate empirical relation between the calculated and the measured percentages of secondary stress. If the various observations are plotted with these percentages as co-ordinates, all but the extreme points are found to be grouped along a straight line of the formula,

$$\text{Percentage of measured secondary stress} = \frac{1}{2}\,(\text{percentage of calculated secondary stress}) + 15.$$

This expression may be interpreted as follows:

1.—The factor, $\frac{1}{2}$, was the average ratio of actual to calculated stresses, and represents the reduction of the bending strains by the yielding of the joints and the internal readjustment of the structure to relieve the secondary stresses. This factor would probably have a higher value in other structures.

2.—The absolute term, 15%, represents additional secondary strains due to factors not included in the computations, such as inaccuracies of fabrication, effects of temperature, dead weight of members, eccentric bearings, etc. This term would probably have a lower value in other structures.

For the most instructive comparison, it is necessary to average the calculated and measured values in groups corresponding to the successive erection stages. This yields the results shown in Table 5.

TABLE 5.

Stage.	Average primary stress.	Average Secondary Stress.	
		Calculated.	Measured.
2	612	1 220	485
4	1 370	703	547
6	2 040	847	796
8	1 254	1 335	711
10	2 393	1 629	725
3-*H*	5 820	557	832
2-*H*	11 620	513	1 140 (296)

The values in Table 5 are plotted in graphs in Fig. 5. Three curves are shown: primary stresses, calculated secondary stresses, and measured secondary stresses.

The curve of primary stresses shows a large drop from Stage 6 to Stage 8, representing the reduction in stresses when the load was transferred from the lower to the upper back-stay.

The curve of calculated secondary stresses is rather high in the first two stages. This is simply because at these stages only one or two panels were erected, and these end panels usually have higher secondary stresses than the intermediate panels of the span. From Stage 4 to Stage 12, the calculated secondary stresses increase continuously with the increasing deflections. At Stage 3-*H* there is a sudden drop, as the 3-hinged condition has very small deflections and, consequently, low secondary stresses.

The measured secondary stresses for the first few stages are very considerably below the calculated values. This proves that the three-faced joints (Fig. 6) acted partly as hinges which relieved the secondary stresses. As the direct stress increased, however, the joints were compressed, the bearing area was enlarged, and the drift-pins in the outer thirds of the joint began to take stress. This accounts for the increasing slope of the curve as Stage 6 is approached, and the diminishing difference between calculated and measured values. From Stage 6 to Stage 8 there is a decline in the measured secondary stresses due to the temporary release in the direct compression at the joints, combined with a reversal of flexure at some of the joints during this transition. Beyond Stage 8 the measured secondary stresses increase continuously.

The final drop in the calculated stress curve is not duplicated in the curve of measured secondary stresses. In the former curve, the

strains for Stages 12 and 3-*H* are the results of independent computations for two entirely distinct erection conditions, and this accounts for the break in the curve. In the measured secondary stresses, how-

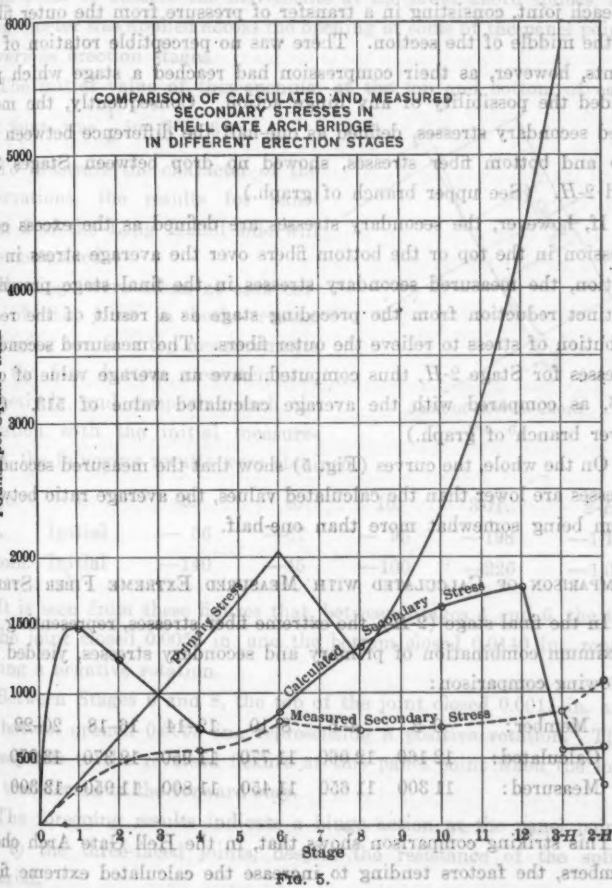


FIG. 5.

ever, there is a continuity of operation, and therefore of stress conditions. The joints are fixed under the large direct compressions, so that some of the secondary stresses induced under the preceding stages persist in the structure when Stages 3-*H* and 2-*H* are reached.

The stresses in Stage 3-*H* were measured before the joints were riveted. During the operation of riveting, as the drift-pins, one by one, were replaced by rivets, there was a gradual redistribution of stresses at each joint, consisting in a transfer of pressure from the outer fibers to the middle of the section. There was no perceptible rotation of the joints, however, as their compression had reached a stage which precluded the possibility of any hinge action. Consequently, the measured secondary stresses, defined as one-half the difference between the top and bottom fiber stresses, showed no drop between Stages 3-*H* and 2-*H*. (See upper branch of graph.)

If, however, the secondary stresses are defined as the excess compression in the top or the bottom fibers over the average stress in the section, the measured secondary stresses in the final stage present a distinct reduction from the preceding stage as a result of the redistribution of stress to relieve the outer fibers. The measured secondary stresses for Stage 2-*H*, thus computed, have an average value of only 296, as compared with the average calculated value of 513. (See lower branch of graph.)

On the whole, the curves (Fig. 5) show that the measured secondary stresses are lower than the calculated values, the average ratio between them being somewhat more than one-half.

Comparison of Calculated with Measured Extreme Fiber Stress.

In the final stage (2-*H*), the extreme fiber stresses, representing the maximum combination of primary and secondary stresses, yielded the following comparison:

Member:	0–2	4–6	8–10	12–14	16–18	20–22
Calculated:	12 160	12 060	11 770	11 950	12 370	13 270
Measured:	11 300	11 650	11 450	11 800	11 950	13 300

This striking comparison shows that, in the Hell Gate Arch chord members, the factors tending to increase the calculated extreme fiber stresses and those tending to reduce them very nearly balance each other. Were it not for the three-faced butt joints and the deferred riveting, the measured extreme stresses would have exceeded the calculated values.

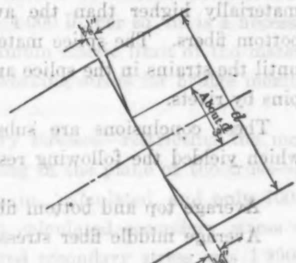
Other Applications of the Extensometer.

In order to observe the movements at the lower chord splices, the extensometer was applied across the opening at some of the panel points in various erection stages.

The initial value of this opening, at the top and bottom of each butt joint, was $\frac{1}{8}$ in. (Fig. 6).

To illustrate the character of the observations, the results for panel point 2 of the Long Island side will be summarized.

Readings were taken at each corner of the joint in both trusses. Averaging the four top measurements and the four bottom measurements, respectively, and comparing each observation with the initial measurement, the following results were obtained:



Joint of Bottom Chord

Fig. 6.

Stage.	4.	6.	8.	10.	3-*H*.	2-*H*.
Top.	Initial	—56	—67	—95	—198	—1 186
Bottom.	Initial	—140	—45	—100	—226	—1 050

It is seen from these figures that, between Stages 4 and 6, the top of the joint closed 0.0056 in. and the bottom closed 0.0140 in., representing a negative rotation.

Between Stages 6 and 8, the top of the joint closed 0.0011 in. and the bottom opened 0.0095 in., representing a positive rotation. This corresponds to the reversed flexure at this panel point when the load was transferred to the forward stay.

The foregoing results indicate a hinge action at the panel points due to the three-faced joints, despite the resistance of the splice material.

The closing of the joints, at top and bottom, took place mainly between Stages 3-*H* and 2-*H*, during which stages the splices were riveted, which indicates a gradual re-adjustment of stress at the joint as the drift-pins were replaced one by one by rivets.

Similar results were obtained at the other panel points observed.

In every case there was a movement of nearly $\frac{1}{8}$ in. from the initial measurement, indicating practically a complete closing of the joint.

This closing of the joints implies a greater compression, by $\frac{1}{8}$ in., in the middle fibers than in the outer fibers of the member. As a result, the average stress over the middle third of the section tends to be materially higher than the average stress in the extreme top and bottom fibers. The splice material across the joints resists this effect until the strains in the splice are released by the replacing of the drift-pins by rivets.

These conclusions are substantiated by the final measurements, which yielded the following results:

	Stage 3-*H*.	Stage 2-*H*.
Average top and bottom fiber stress......	6 050	10 400
Average middle fiber stress...............	6 050	13 100

The final difference between extreme and middle fiber stress thus amounts to 2 700 lb. per sq. in. at the sections of measurement. The gauge points were approximately at the quarter points of the length between panel points; a larger difference, of course, would be found if the sections could be taken nearer to the ends of the members.

SUMMARY OF CONCLUSIONS.

1.—The Howard strain gauge is well adapted for the measurement of stresses in steel structures under quiescent load, provided that the stresses are not less than 1 000 lb. per sq. in. With careful manipulation it is possible to determine the true stresses within less than 200 lb. per sq. in.

2.—A comparison between calculated and measured primary stresses reveals, in addition to the observational errors, an average difference of 5 per cent. This is due principally to variations in the loading and cross-sections from the values assumed, and indicates the futility of excessive refinement in the ordinary computation of stresses.

3.—The extreme variation of fiber stress from the average stress in a member was found to range from about 1 600 lb. per sq. in. with the lowest primary stresses to about 2 500 with the highest primary stresses. A part of this variation represented the secondary stress from vertical bending; the greater part, however, was due to the effects of lateral bending, shop inaccuracies, temperature strains, splice details,

non-uniform material, and other causes which are omitted from consideration in the computation of secondary stresses.

4.—Combining the maximum discrepancy between calculated and measured stress (5% + 1 000) with the maximum variation of extreme fiber stress (12% + 2 600), and allowing for the fact that in the Hell Gate Arch the three-faced joints tended to reduce the secondary stresses, it appears that about 25% + 4 000 lb. per sq. in. is a necessary margin to be deducted from the minimum elastic limit of the material in order to obtain the limiting safe working stress for bridge members under average conditions.

5.—During erection, the secondary stresses—restricting the meaning of the term to the effect of bending in the plane of the truss—had an average value of 1 050 lb. per sq. in., calculated, and only 700 lb. per sq. in., measured. The highest calculated secondary stress was 2 920, and the corresponding measured secondary stress was 1 990 lb. per sq. in. Except for the smallest secondary stresses, the measured values were consistently lower than the calculated values. For the highest percentages of secondary stresses, the measured values are only a small fraction of the calculated values.

It is believed that similar results, though not as marked, would be found in other structures. The actual secondary stresses will generally be lower than the calculated values. There is an automatic re-adjustment of strains within a structure in such direction as to relieve the secondary stresses.

6.—The variations of calculated and measured secondary stresses in the successive erection stages are plotted for comparison in Fig. 5. The graphs show the measured stresses lower than the calculated values throughout the erection. The differences between the two curves are explained by special conditions in the erection of the structure.

7.—In the early erection stages, the three-faced joints (Fig. 6) between the lower chord members acted as hinges to permit a certain amount of rotation, so as to ease the secondary stresses. In the succeeding stages, as the direct stress increased, the joints became compressed, and the rotation was restricted. Between Stages 6 and 8, when there was a decrease in the direct compression, rotation occurred again, and the secondary stresses were partly released.

Extensometer measurements across the splice openings confirmed the above-described hinge action of the joints.

Another object of the three-faced joints was to produce a concentration of pressure in the middle third of the section, accompanied by a reduction of direct stress in the outer fibers. This distribution of stress, with the largest intensities in the middle third, was confirmed by actual measurement in the final stage.

8.—Measurements across the splice openings (Fig. 6) before and after riveting showed a complete closing of the joints during this operation. This is proof of the release of strains at a joint when the drift-pins are replaced by rivets.

The closing of the openings represents a desired transfer of initial stress from the splice material to the butt joint; it is also visible evidence of the greater concentration of stress in the middle third of the cross-section.

9.—The curve of calculated secondary stresses shows a large drop from the cantilever to the three-hinged stage, on account of the large reduction in deflections. This indicates the comparative freedom of the arch type from secondary stresses.

10.—In the final stage, the stresses in the extreme fibers, representing the maximum combined effect of primary and secondary stresses, show a remarkable agreement with the calculated values of the same stresses.

Acknowledgments.

Acknowledgments, in addition to those already given, are due to Mr. Frank E. Berry and to Theodore Belzner, Assoc. Am. Soc. C. E., for instruction and suggestions in the use of the extensometer; to Messrs. Brown and Sharpe, manufacturers of the instrument, for courtesies extended; to O. H. S. Koch, Jun. Am. Soc. C. E., who, with assistants, took all the measurements, and on whose painstaking thoroughness their value depended; to Mr. F. de Schauensee, who calculated the secondary stresses and assisted in reducing the observations; and to O. H. Ammann, M. Am. Soc. C. E., for suggestions during the prosecution of the investigation.

APPENDIX

SECONDARY STRESSES IN ARCH TRUSSES OF HELL GATE BRIDGE.

In view of the unusual size and form of the trusses of the Hell Gate Arch Bridge and for comparison with stress measurements, a complete analysis was made of the secondary stresses from dead load, live load, and temperature changes, in the finished structure, as well as during the various stages of erection.

The writer selected Winkler's analytical method, for these computations, in preference to the graphic method of Professor Mohr, because the greater precision and ease of supervision appeared to be sufficient compensation for the slight increase in time required. Except for some changes in the arrangement and tabulation of the computations, the procedure given in Johnson, Bryan, and Turneaure's "Modern Framed Structures" was closely followed. The calculation of secondary stresses for each load condition involves the solution of a series of simultaneous equations equal in number to the total number of panel points in the truss, and the expediting of the entire computation depends largely on the method selected for solving these equations. The quickest and best results were obtained with a modification of the method of successive approximations described by F. E. Turneaure, M. Am. Soc. C. E.* The modification consisted in substituting, in the first and each successive solution, the new values of the unknowns as far as already obtained, instead of using the values from the preceding approximation.

The computation of the secondary stresses from dead load on the three-hinged arch were simplified by treating the two halves of the arch, up to the temporary crown-hinge, as separate frames. The effect of friction at the hinges was neglected.

For the dead-load secondary stresses in the two-hinged arch, because of symmetry, only one-half of the truss had to be computed. The load for this case consisted of the concrete floor and tracks, which were added after the center top chord was connected.

In computing the secondary stress for live load, one-half of the arch was considered loaded, as this load produces nearly the maximum primary stress in most members. A simple reversal of the diagram and algebraic addition of the two sets of stresses gives the secondary stresses for live load covering the full span.

The results of the secondary stress computations for the Hell Gate Arch are recorded on Plates XLV, XLVI, and XLVII. These plates also contain all data necessary for reproducing the computations.

* *Engineering News*, September 5th, 1912.

The diagrams give, for each designated condition of loading, the secondary stresses at both ends of each member. A double sign prefixed to each figure indicates the kind of stress in the top and bottom fibers of the section, respectively. In addition, the primary stress for the same condition of loading is marked on each member in parentheses.

Another feature in these diagrams is the graphic representation, for the individual members, of the deformations corresponding to the secondary stresses. These deformations are shown exaggerated to an arbitrary relative but non-proportional scale. After the curves are drawn they serve principally as a general indication of relative distortions. In addition, the sharpness of curvature at any point represents the intensity of bending moment; the points of contraflexure mark points of zero secondary stress; the direction of curvature gives the signs of the secondary stress; the configuration of curves meeting at any panel point determines the direction of deflection of that point; and, in general, the deformation curves afford a visual check on the correctness of the work.

RESULTS OF THE SECONDARY STRESS CALCULATIONS.

An inspection of the secondary stresses recorded on Plate XLV yields the following facts:

The largest dead-load secondary stresses, in pounds per square inch, are as follows:

Lower Chord.	Upper Chord.	Diagonals.	Verticals.
1 290 in 20-22	2 468 in 1-3	4 950 in 22-*M*	2 186 in 20-21
1 032 in 18-20	961 in 21-23	1 376 in 3-4	1 671 in 18-19
		1 316 in 1-2	
All the other stresses are below			
800	800	1 300	1 600

It will be observed that the largest secondary stresses in each class of members occur in the end panels and crown panels of the span. This effect is accounted for by the large concentrations of stress at Panel Points 0 and 22 of the three-hinged arch. It will generally be found that the largest secondary stresses in any structure occur where there is an interruption in the continuity of the truss configuration, or in the uniformity of the loading conditions.

Plate XLV also affords a comparison of the secondary stresses in a three-hinged and a two-hinged arch. The former are generally the larger.

In the outstanding dead-load secondary stresses it will be found that the major contributions to the total stress come from the three-hinged condition. This effect is most marked in the panels near the temporary crown-hinge, as shown by Table 6.

PLATE XLV.
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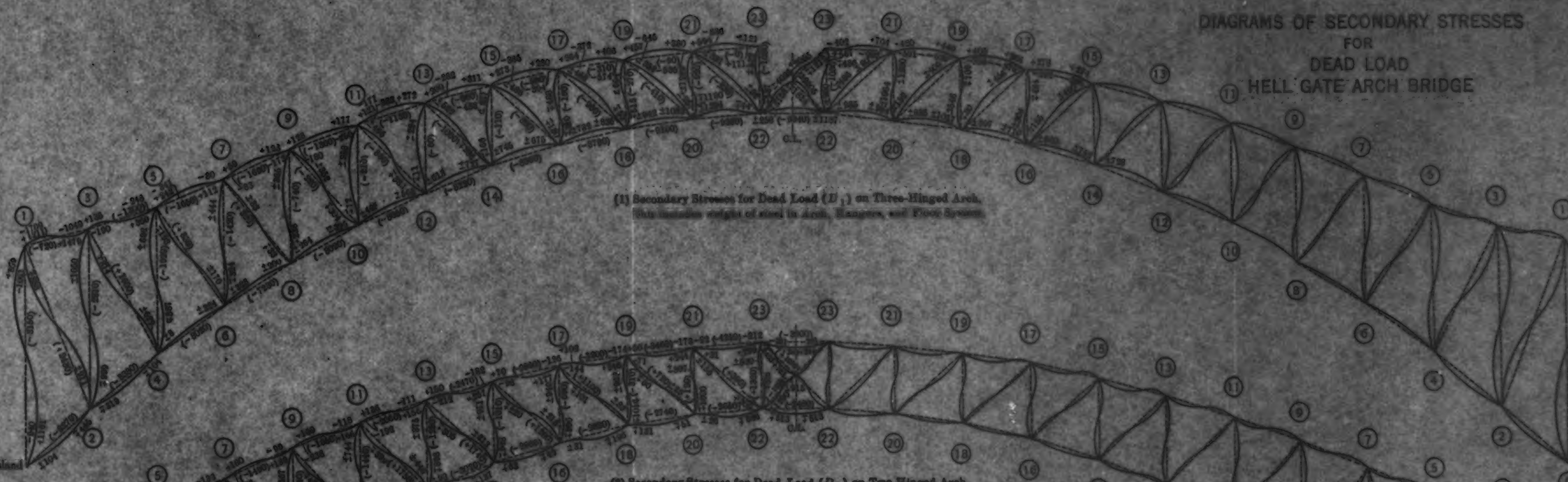
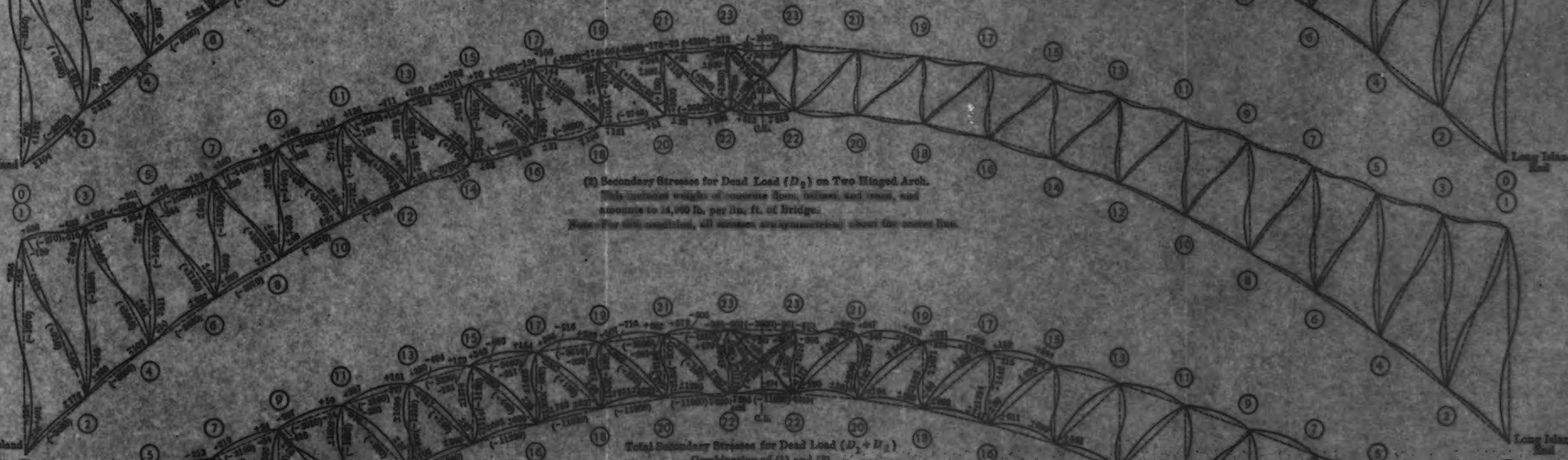
STRESS MEASUREMENTS,
HELL GATE ARCH BRIDGE.

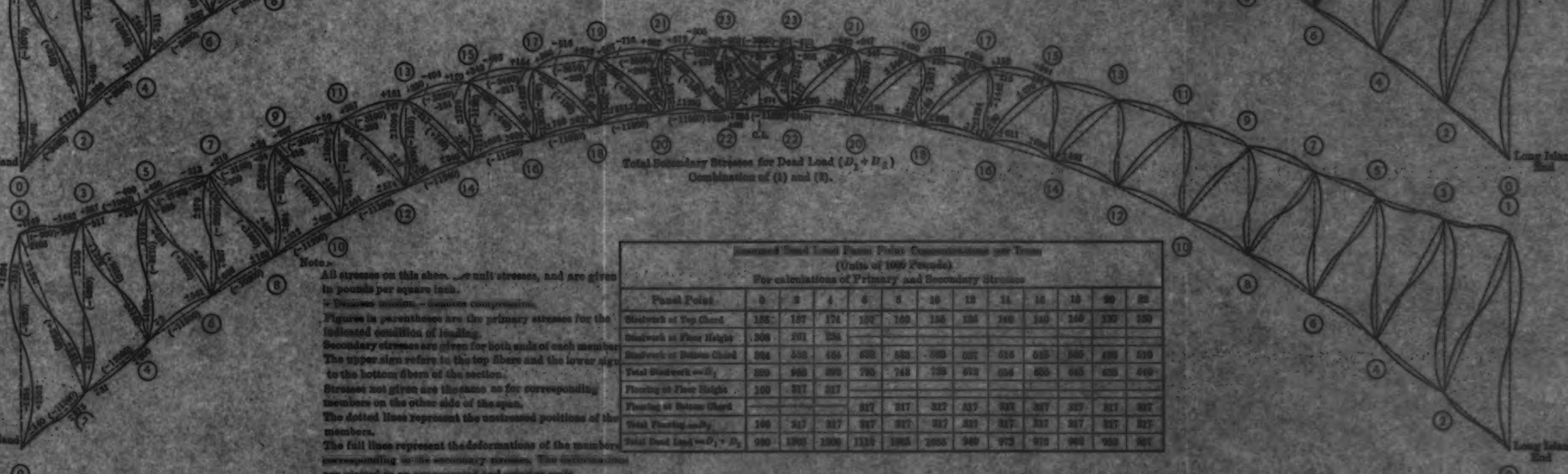
DIAGRAMS OF SECONDARY STRESSES FOR DEAD LOAD HELL GATE ARCH BRIDGE



(1) Secondary Stresses for Dead Load (D_1) on Three-Hinged Arch.
This includes weight of steel in Arch, Hangers, and Floor System.



(2) Secondary Stresses for Dead Load (D_2) on Two Hinged Arch.
This includes weight of concrete floor, ballast, and track, and amounts to 14,000 lb. per lin. ft. of Bridge.
Note: For this condition, all stresses are symmetrical about the center line.

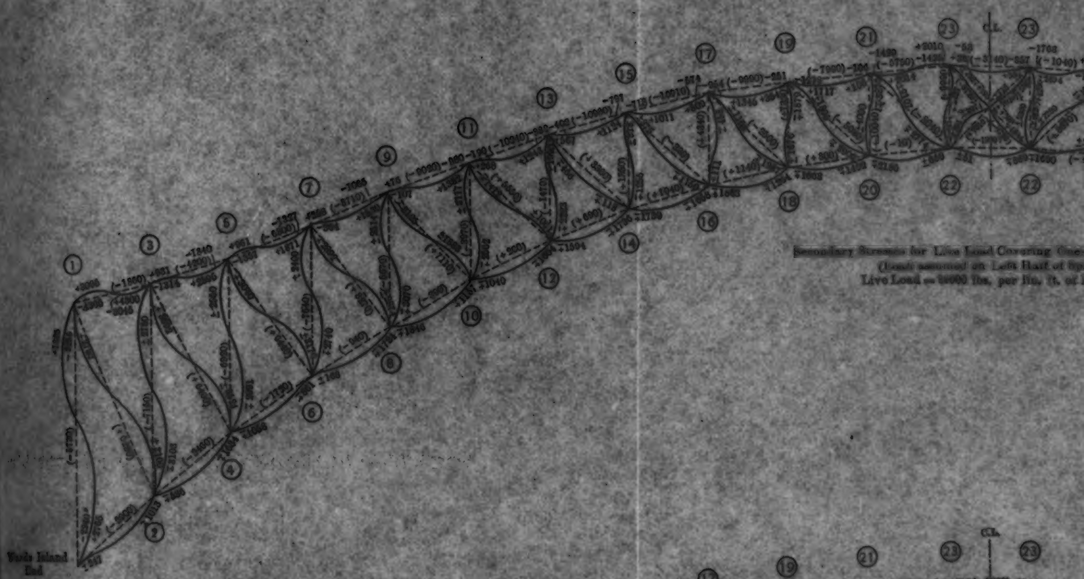


Total Secondary Stresses for Dead Load ($D_1 + D_2$)
Combination of (1) and (2).

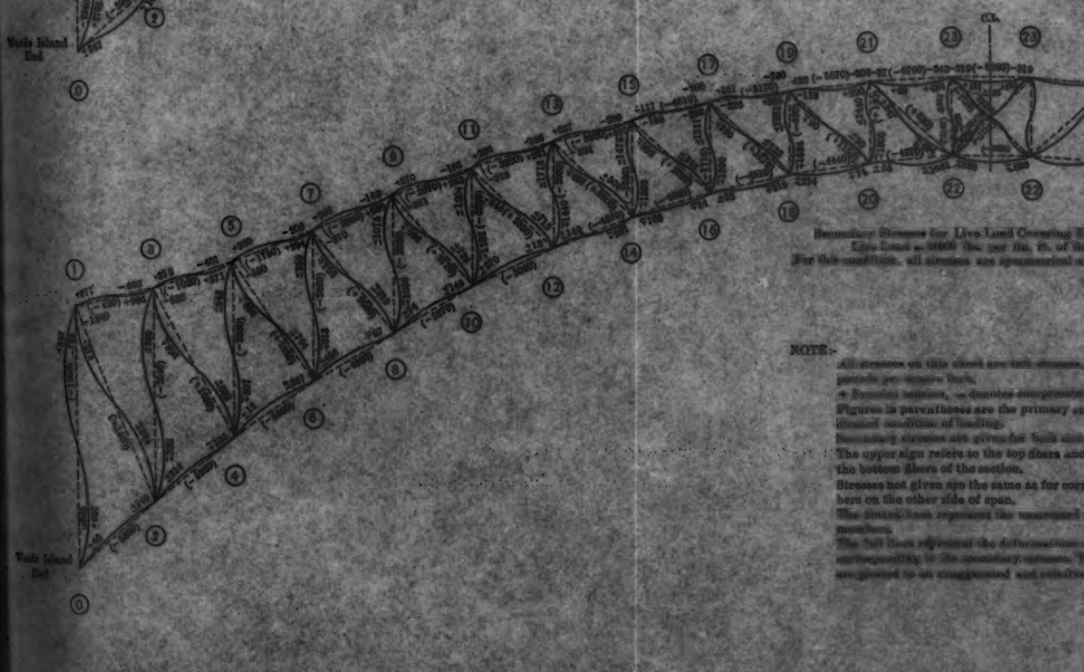
Note:-
All stresses on this sheet are unit stresses, and are given in pounds per square inch.
+ Denotes tension, − denotes compression.
Figures in parentheses are the primary stresses for the indicated condition of loading.
Secondary stresses are given for both ends of each member. The upper sign refers to the top fibers and the lower sign to the bottom fibers of the section.
Stresses not given are the same as for corresponding members on the other side of the span.
The dotted lines represent the unstressed positions of the members.
The full lines represent the deformations of the members corresponding to the secondary stresses. The deformations are plotted to an exaggerated and relative scale.

Assumed Dead Load Panel Point Concentrations per Truss
(Units of 1000 Pounds)
For calculations of Primary and Secondary Stresses

Panel Point	0	2	4	6	8	10	12	14	16	18	20	22
Steelwork at Top Chord	188	187	174	157	160	156	136	140	140	140	137	130
Steelwork at Floor Height	308	261	236	—	—	—	—	—	—	—	—	—
Steelwork at Bottom Chord	324	538	484	638	583	583	537	516	515	505	498	510
Total Steelwork = D_1	820	988	898	795	743	738	672	656	655	645	635	640
Flooring at Floor Height	160	317	317	—	—	—	—	—	—	—	—	—
Flooring at Bottom Chord	—	—	—	317	317	317	317	317	317	317	317	317
Total Flooring = D_2	160	317	317	317	317	317	323	317	317	317	317	317
Total Dead Load = $D_1 + D_2$	980	1305	1300	1112	1065	1055	989	973	972	962	952	957



Secondary Stresses for Live Load Covering One-
(Load assumed on Left Half of Sp
Live Load = 18000 lbs. per lin. ft. of



Secondary Stresses for Live Load Covering E
Live Load = 18000 lbs. per lin. ft. of B
For this condition, all stresses are symmetrical a

NOTE:-

All stresses on this sheet are unit stresses,
pounds per square inch.
+ Denotes tension, − denotes compressio
Figures in parentheses are the primary st
dicated condition of loading.
Secondary stresses are given for both ends
The upper sign refers to the top fibers and
the bottom fibers of the section.
Stresses not given are the same as for corr
bers on the other side of span.
The dotted lines represent the unstressed
members.
The full lines represent the deformations
corresponding to the secondary stresses. T
are plotted to an exaggerated and relative

PLATE XLVI.
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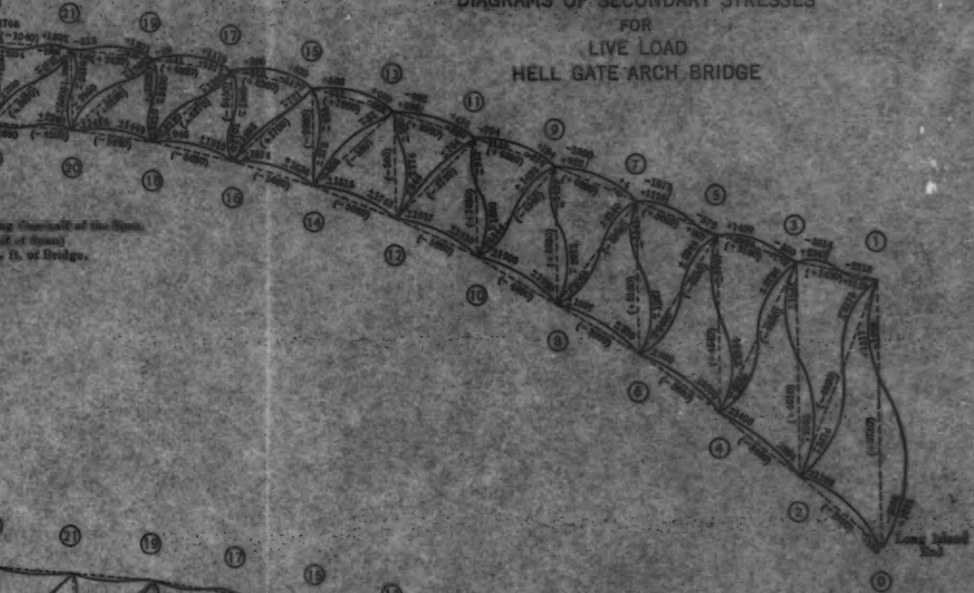
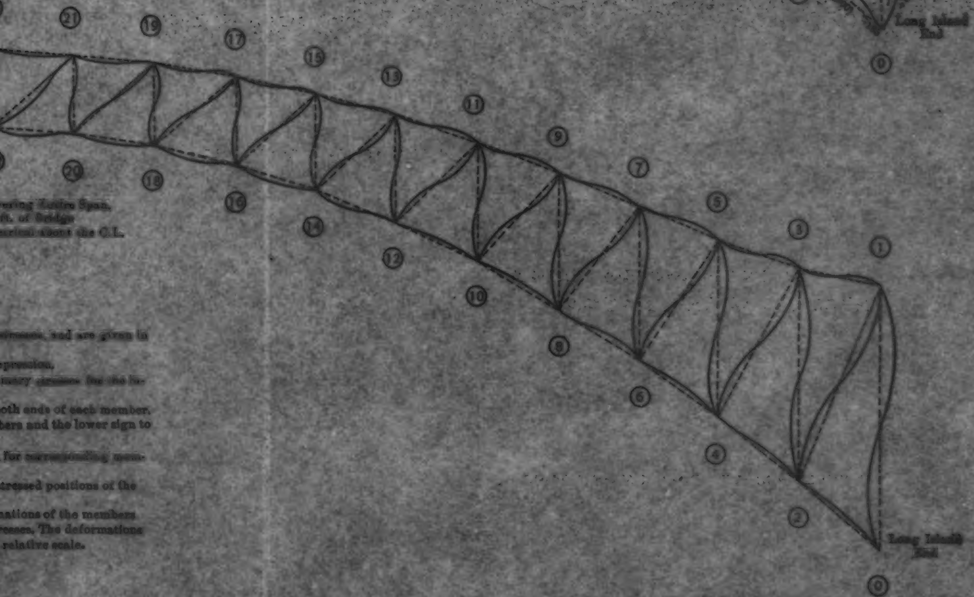
STEINMAN ON
STRESS MEASUREMENTS,
HELL GATE ARCH BRIDGE.

DIAGRAMS OF SECONDARY STRESSES FOR LIVE LOAD HELL GATE ARCH BRIDGE





In units of 1000 lb.

	Member	Length (In.)	Cross Section (In.²)	Moment of Inertia (In.⁴)	Extreme Fiber Distance (In.)	(1) Dead Load: Wards L. Side		Long L. Side		(2) Live Load Covering One-half Span: Loaded Side		Unloaded Side		(3) Live Load Covering Whole Span		(4) Approximate Secondary Stress Occurring Simultaneously with Max. Live Load Stress		(5) Total (1)+(4): Wards L. Side		Long L. Side		Primary Stress Corresponding To (1)	(2)	(3)	Per cent Secondary Stress of Primary Stress (1)	(2)	(3)
Lower Chord	0-2	674	1884	2219000	65.0	± 140	∓ 273	± 140	∓ 273	± 247	∓ 1013	∓ 183	± 1162	± 63	± 149	± 68	± 149	± 203	∓ 124	± 203	∓ 124	-11890	-5800	-17690	2	3	1
	2-4	685	1281	1988000	61.4	∓ 31	± 206	∓ 31	∓ 206	∓ 636	∓ 1034	± 564	± 1498	± 816	∓ 150	± 382	± 60	± 351	∓ 146	± 351	∓ 146	-12080	-6030	-18110	2	6	2
	4-6	694	1265	1857000	59.6	∓ 2	∓ 561	∓ 2	± 561	∓ 1056	∓ 902	± 1010	± 568	± 12	± 387	± 300	± 447	± 298	± 1003	± 298	± 1003	-11500	-6070	-17570	5	7	6
	6-8	608	1227	1943000	56.4	± 606	± 158	± 606	± 158	± 163	∓ 1734	± 396	± 1607	± 458	∓ 27	± 396	± 629	± 1004	± 816	± 1004	± 816	-10300	-5700	-16030	6	11	6
	8-10	542	1163	1461000	56.4	± 312	± 462	± 312	± 462	∓ 1846	∓ 1151	± 1920	± 1303	± 74	± 146	± 961	± 682	± 1243	± 1164	± 1243	± 1164	-11290	-6570	-17[illegible]	4	14	7
	10-12	564	1121	1250000	52.6	± 591	± 574	± 591	± 574	∓ 1040	∓ 1604	± 1810	± 1617	± 270	± 13	± 828	± 874	± 1419	± 1448	± 1419	± 1448	-11190	-6680	-17570	5	13	9
	12-14	543	1080	1058000	49.4	± 706	± 590	± 706	± 591	∓ 1084	∓ 1736	± 1748	± 1515	± 149	∓ 281	± 1104	± 818	± 1810	± 1408	± 1810	± 1409	-11240	-6670	-17910	6	17	10
	14-16	535	1046	971000	47.5	± 632	± 620	± 699	± 611	∓ 1739	∓ 1659	± 1696	± 1574	∓ 108	∓ 84	± 1037	± 1004	± 1719	± 1680	± 1720	± 1615	-11230	-6410	-17630	6	16	10
	16-18	531	989	861000	45.5	± 763	± 687	± 741	± 699	∓ 1663	∓ 1304	± 1913	± 946	± 59	∓ 316	± 1345	± 559	± 2106	± 1196	± 2086	± 1283	-11610	-6090	-17600	7	22	12
	18-20	516	946	764000	43.4	± 611	± 1002	± 686	± 784	∓ 1668	∓ 1492	± 1449	± 1415	∓ 214	∓ 74	± 582	± 595	± 1346	± 1627	± 1418	± 1370	-11680	-5400	-17580	9	11	8
	20-22	512	929	660000	41.9	± 1290	∓ 660	± 1010	± 290	∓ 2165	± 680	± 2240	∓ 1000	± 56	∓ 1031	± 589	∓ 1160	± 1670	∓ 1888	± 1560	∓ 890	-11980	-4920	-16900	11	24	11
	22-22.4	510	929	690000	41.9	∓ 256	± 674	± 674	∓ 256	± 51	∓ 889	∓ 889	± 51	∓ 888	∓ 888	∓ 848	∓ 690	∓ 1102	∓ 22	∓ 172	∓ 221	-11620	-[illegible]	-16740	6	17	7
Upper Chord	1-3	523	315	98000	10.5 27.5	+ 1749 − 2468	− 1482 + 2080	+ 1749 − 2468	− 1482 + 2038	+ 3035 − 4365	− 3045 + 4300	− 2216 + 3125	+ 2347 − 3814	+ 877 − 1240	− 698 + 986	+ 2215 − 3130	− 2180 + 3060	+ 3964 − 5606	− 3662 + 5166	+ 3964 − 5606	− 3662 + 5166	− 980	− 2114	− 3104	177 250	105 148	128 180
	3-5	535	315	98000	10.5 27.5	+ 367 − 517	− 499 + 704	+ 367 − 517	− 499 + 704	+ 931 − 1314	− 1840 + 2596	− 560 + 788	+ 1430 − 2024	+ 372 − 526	− 401 + 571	+ 696 − 980	− 1371 + 1931	+ 1062 − 1497	− 1870 + 2635	+ 1062 − 1497	− 1870 + 2635	− 1980	− 4680	− 6630	28 36	29 41	28 40
	5-7	560	315	98000	10.5 27.5	+ 486 − 685	− 213 + 299	+ 486 − 685	− 213 + 299	+ 961 − 1357	− 1327 + 1871	− 574 + 807	+ 1119 − 1577	+ 389 − 560	− 206 + 294	+ 740 − 1061	− 1036 + 1462	+ 1226 − 1738	− 1243 + 1751	+ 1226 − 1738	− 1242 + 1751	− 2720	− 7420	− 10140	13 25	14 20	12 17
	7-9	560	330	110000	10.5 27.5	+ 219 − 309	+ 24 − 34	+ 219 − 309	+ 24 − 34	+ 258 − 364	− 1086 + 1530	+ 4 − 0	+ 922 − 1300	+ 262 − 370	− 182 + 260	+ 210 − 295	− 881 + 1241	+ 429 − 604	− 857 + 1207	+ 429 − 604	− 857 + 1207	− 3180	− 9330	− 12510	7 10	9 13	7 10
	9-11	543	386	110000	10.5 27.5	+ 297 − 418	+ 59 − 84	+ 297 − 418	+ 59 − 84	+ 76 − 107	− 966 + 1362	+ 104 − 274	+ 794 − 1119	+ 270 − 381	− 156 + 203	+ 64 − 90	− 825 + 1169	+ 361 − 508	− 709 + 1058	+ 361 − 508	− 709 + 1046	− 3010	− 9430	− 12440	10 14	9 12	6 9
	11-13	535	386	110000	10.5 27.5	+ 257 − 363	+ 161 − 227	+ 257 − 362	+ 161 − 227	− 190 + 208	− 882 + 1243	+ 416 − 580	+ 696 − 982	+ 226 − 318	− 186 + 261	− 170 + 240	− 788 + 1111	+ 87 − 122	− 627 + 884	+ 87 − 122	− 627 + 884	− 3190	− 10400	− 13590	8 11	8 11	5 6
	13-15	537	386	110000	10.5 27.5	+ 330 − 494	+ 129 − 181	+ 280 − 494	+ 129 − 181	− 402 + 567	− 797 + 1124	+ 630 − 891	+ 508 − 717	+ 237 − 334	− 289 + 407	− 378 + 534	− 786 + 1109	− 38 + 46	− 621 + 873	− 38 + 46	− 621 + 870	− 3330	− 11040	− 14370	11 15	7 10	4 6
	15-17	541	386	110000	10.5 27.5	+ 343 − 482	+ 154 − 217	+ 344 − 484	+ 152 − 215	− 718 + 1011	− 574 + 809	+ 835 − 1179	+ 366 − 516	+ 117 − 166	− 208 + 293	− 718 + 1011	− 574 + 809	− 375 + 528	− 420 + 592	− 374 + 527	− 430 + 591	− 3440	− 10860	− 14340	10 14	7 9	3 4
	17-19	516	386	110000	10.5 27.5	+ 366 − 510	+ 232 − 328	+ 368 − 519	+ 231 − 324	− 954 + 1346	− 251 + 354	+ 1115 − 1571	− 29 + 41	+ 161 − 226	− 289 + 396	− 885 + 1247	− 269 + 378	− 519 + 731	− 37 + 52	− 517 + 732	− 38 + 54	− 2810	− 10180	− 13000	10 13	9 12	4 5
	19-21	51.	386	110000	10.5 27.5	+ 507 − 716	+ 207 − 291	+ 490 − 691	+ 247 − 347	− 1218 + 1717	+ 106 − 150	+ 1591 − 1685	− 312 + 440	+ 83 − 118	− 206 + 290	− 1062 + 1497	+ 69 − 97	− 555 + 781	+ 276 − 388	− 572 + 806	− 316 + 444	− 3580	− 8860	− 12440	14 20	12 17	5 6
	21-23	511	330	110000	10.5 27.5	+ 572 − 805	− 343 + 479	+ 682 − 961	− 614 + 866	− 1429 + 2014	+ 1425 − 2010	+ 1396 − 1966	− 1768 + 2494	− 37 + 48	− 341 + 484	− 404 + 570	+ 283 − 400	+ 168 − 235	− 50 + 70	+ 218 − 301	+ 331 − 446	− 4210	− 8880	− 13090	16 23	5 6	3 4
	23-23.4	510	330	110000	10.5 27.5	− 201 + 283	− 201 + 283	− 201 + 283	− 201 + 283	+ 88 − 52	− 357 + 504	− 357 + 504	+ 88 − 63	− 319 + 401	− 319 + 451	− 137 + 194	− 137 + 194	− 338 + 477	− 338 + 477	− 338 + 477	− 338 + 447	− 3900	− 8140	− 13040	5 7	2 2	3 4
Diagonals	1-2	1340	286	80000	27	∓ 1297	± 1316	∓ 1297	± 1316	∓ 2754	± 3100	± 2175	∓ 3416	∓ 578	± 694	∓ 1870	± 2220	∓ 3267	± 3636	∓ 3267	± [illegible]	+ 3340	+ 7140	+ 10480	39	31	34
	3-4	1080	228	70000	27	∓ 1376	± 1160	∓ 1376	± 1160	∓ 2802	± 3634	± 2796	∓ 2686	∓ 890	± 859	∓ 2790	± 2065	∓ 4166	± [illegible]	∓ 4166	± [illegible]	+ 2610	+ 7100	+ 9710	58	[illegible]	40
	5-6	911	202	52000	24	∓ 624	± 646	∓ 624	± 640	∓ 3040	± 3170	± 2944	∓ 2406	∓ 698	± 764	∓ 1983	± 2070	∓ 2607	± 2716	∓ 2607	± 2716	+ [illegible]	+ 6810	+ 8880	35	30	30
	7-8	818	130	[illegible]	21	∓ 466	± 467	∓ 466	± 467	∓ 2892	± 2574	± 2140	∓ 2064	∓ 752	± 810	∓ 1806	± 1792	∓ 2240	± 2250	∓ 2240	± 2269	+ 1980	+ 9180	+ 11160	34	20	20
	9-10	708	130	[illegible]	21	∓ 78	± 158	∓ 78	± 153	∓ 2125	± 1832	± 1439	∓ 1144	∓ 693	± 730	∓ 1800	± 1160	∓ 1378	± 1303	∓ 1378	± 1303	+ 1560	+ 8640	+ 10140	19	15	14
	11-12	722	130	23500	21	± 96	± 87	± 96	± 51	∓ 1412	± 1061	± 764	∓ 349	∓ 648	± 712	∓ 881	± 663	∓ 786	± 729	∓ 786	± 729	+ 980	+ 8910	+ 9890	11	11	9
	13-14	706	130	23500	21	± 217	± 27	± 217	± 27	∓ 496	∓ 118	∓ 31	± 620	∓ 497	± 508	∓ 224	± 102	± 8	± 129	± 8	± 129	+ 740	+ 8450	+ 9190	29	5	1
	15-16	694	180	30700	21	± 268	± 35	± 270	± 30	± 875	∓ 1460	∓ 727	± 1492	∓ 392	± 348	∓ 318	± 530	∓ 60	± 683	∓ 48	± 580	+ 480	+ 7160	+ 7630	61	7	7
	17-18	695	180	30700	21	± 387	± 20	± 385	± 50	± 1186	∓ 1297	∓ 1330	± 2120	∓ 165	± 138	∓ 1039	± 1030	∓ 872	± 1067	∓ 684	± 1092	+ 180	+ 8800	+ 8980	240	10	15
	19-20	675	180	30000	21	± 292	± 202	± 246	± 187	± 1695	∓ 2900	∓ 1986	± [illegible]	∓ 90	± 28	∓ 1656	± 2450	∓ 1366	± 2748	∓ 1812	± 2627	+ [illegible]	+ 9010	+ 9290	112	27	29
	21-22	664	196	30000	21	± 486	∓ 267	± 646	± 177	± 2107	∓ 2774	∓ 1880	± [illegible]	± 247	∓ 494	∓ 1633	± 2046	∓ 2107	± 1790	∓ [illegible]	± 2222	+ 160	+ 10020	+ 10180	304	20	22
	22-24'	391	130	28400	21	∓ 4456	± 4950	∓ 5104	± 2980	∓ 594	± 2074	∓ 1584	∓ 595	∓ 2088	± 1470	∓ 1984	∓ 592	∓ 6312	± 4212	∓ [illegible]	± 2022	− 1060	− 8720	− 9770	471	22	28
	23-24'	391	130	28400	21	± 403	∓ 705	∓ 750	± 1890	± 2627	∓ 1954	∓ 1792	± 716	± 738	∓ 1233	± 2627	∓ 1464	± 2980	∓ 2718	± 1166	∓ 694	− 1060	− 8720	− 9770	128	29	31
Verticals	0-1	1080	312*	110000*	27.5 24	− 1264 + 1562	+ 1224 − 1515	− 1264 + 1562	+ 1224 − 1515	− 2240 + 2765	+ 1825 − 2255	+ 1675 − 2068	− 1228 + 1517	− 565 + 697	+ 597 − 738	− 1700 + 2106	+ 1390 − 1720	− 2072 + 2667	+ 2614 − 3223	− 2072 + 2667	+ 2614 − [illegible]	− 4220*	− 6400*	− 10620*	30 37	27 33	28 35
	2-3	1285	320*	94000*	27	∓ 1436	± 1566	∓ 1436	± 1566	∓ 3108	± 2780	± 2380	∓ 1945	∓ 738	± 805	∓ 2090	± 2129	∓ 3818	± [illegible]	∓ 3818	± [illegible]	− 4300	− 7780	− 12110	35	31	31
	4-5	1115	235*	70000*	27	∓ 1088	± 1219	∓ 1088	± 1219	∓ 2991	± 2640	± 2184	∓ 1707	∓ 807	± 848	∓ 2442	± 2080	∓ 3540	± 3301	∓ 3540	± 3301	− 3420	− 7420	− 10840	38	33	33
	6-7	970	151	[illegible]	21	∓ 837	± 979	∓ 837	± 979	∓ 2740	± 2400	± 1967	∓ 1541	∓ 813	± 820	∓ 1710	± 1560	∓ 2537	± 2479	∓ 2537	± 2479	− 2960	− [illegible]	− 11[illegible]	38	29	22
	8-9	884	137	[illegible]	21	∓ 894	± 1032	∓ 894	± 1032	∓ 3070	± 2614	± 2012	∓ 1430	∓ 1058	± 1073	∓ 1980	± 1572	∓ 2814	± 2604	∓ 2814	± 2604	− 2160	− [illegible]	− [illegible]	47	28	29
	10-11	761	136	27000	21	∓ 804	± 1003	∓ 804	± 1003	∓ 2562	± 2117	± 1380	∓ 913	∓ 1182	± 1284	∓ 1151	± 1008	∓ [illegible]	± [illegible]	∓ [illegible]	± 2008	− 1630	− [illegible]	− [illegible]	61	16	22
	12-13	682	136	27000	21	∓ 900	± 1099	∓ 900	± 1099	∓ 2228	± 1902	± 774	∓ 280	∓ 1449	± 1715	∓ 738	± 808	∓ 1647	± [illegible]	∓ 1647	± 1896	− [illegible]	− 7440	− 8770	84	11	22
	14-15	615	136	27000	21	∓ 1114	± 1278	∓ 1114	± 1278	∓ 1226	± 818	∓ 378	± 655	∓ 1633	± 1473	∓ 664	± 900	∓ 1768	± [illegible]	∓ 1768	± 2174	− 1180	− 8410	− 9590	114	11	23
	16-17	562	136	27000	21	∓ 1300	± 1498	∓ 1394	± 1484	± 111	∓ 458	∓ 1701	± 1960	∓ 1500	± 1611	∓ 1673	± 1875	∓ 2972	± [illegible]	∓ 2967	± 3274	− 900	− 9700	− 10610	162	19	31
	18-19	562	136	27000	21	∓ 1531	± 1056	∓ 1562	± 1671	± 1676	∓ 1679	∓ 3310	± 2430	∓ 1534	± 1631	∓ 2029	± [illegible]	∓ 4580	± 4769	∓ 4591	± 4784	− 540	− 10710	− 11250	[illegible]	28	[illegible]
	20-21	486	130	27000	21	∓ 2180	± 2180	∓ 2064	± 2127	± 4000	∓ 3870	± 5005	∓ 5310	∓ 1605	± 1440	± 3264	∓ 2540	± 1484	∓ 1394	± 1404	∓ 1368	− 900	+ 11[illegible]	+ 10[illegible]	213	22	15
	22-23	428	136	27000	21	∓ 1115	± 1032	∓ 1038	± 1014	± 4274	∓ 3427	∓ 5200	± 4890	∓ 926	± 883	± 4874	∓ [illegible]	∓ [illegible]	∓ [illegible]	± [illegible]	∓ 2411	+ [illegible]	+ [illegible]	+ [illegible]	[illegible]	[illegible]	15

* Average Value

PLATE XLVII.
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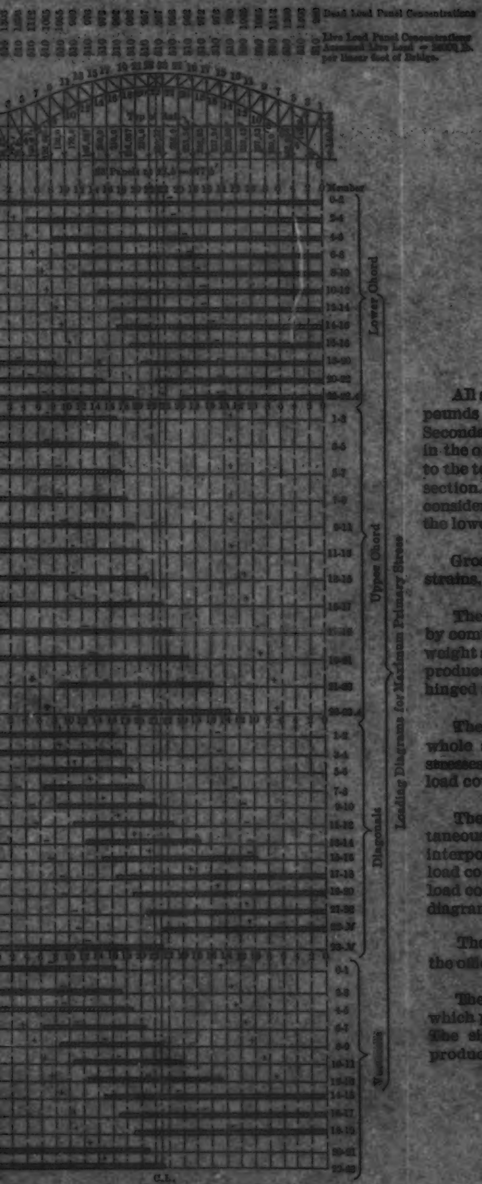


—The above loading diagrams refer to members in the left half of span only. For corresponding members in the right half of span, the loading diagrams should be reversed.

TABLE OF SECONDARY STRESSES HELL GATE ARCH BRIDGE.

NOTES

All stresses on this sheet are unit stresses, and are given in pounds per sq. in. + denotes tension, — denotes compression. Secondary stresses are given for both ends of each member in the order of the designating numbers. The upper sign refers to the top fibers and the lower sign to the bottom fibers of the section. In verticals the fibers toward the springing are considered as the upper fibers and those toward the crown as the lower fibres.

Gross sections were used for figuring unit stresses and strains.

The secondary stresses ① from dead load were obtained by combining the secondary stresses produced by the steel weight acting on the 3-hinged arch with the secondary stresses produced by the weight of floor and track acting on the 2-hinged arch.

The secondary stresses ③ from live load covering the whole span, were obtained by combining the secondary stresses ② in the loaded and unloaded sides produced by live load covering one-half of the span.

The approximate secondary stresses ④ occurring simultaneously with maximum live-load stress were obtained by interpolation between the secondary stresses ② from live load covering one-half of the span and those ③ from live load covering the whole span, in accordance with the loading diagrams appended to the table.

The primary stresses given on this sheet were figured from the official Table of Stresses for the Hell Gate Arch.

The loading diagrams indicate by shaded areas the loading which produces maximum primary stress in each member. The signs (+) or (—) indicate the character of the stress produced by any loads in the shaded or unshaded distances.

TABLE 6.

Member.	Secondary stress from dead load on 3-hinged arch, in pounds per square inch.	Total secondary stress from dead load, in pounds per square inch.
18-20	1 083	1 032
20-22	1 264	1 290
21-23	992	961
22-M	4 035	4 950
M-23	2 055	1 290

The loading added in the two-hinged condition, although amounting to nearly one-half of the dead load on the three-hinged arch, does not produce its proportional share of the total secondary stress. In many cases it even helps to neutralize or reduce the secondary stress originating in the three-hinged condition.

An inspection of the secondary stresses on Plate XLVI yields the following facts:

As was to be expected from the respective deformations, the secondary stresses for live load covering the half span are considerably greater than for the live load covering the full span.

With only two or three exceptions in each case, the secondary stresses for full live load are less than 400 in the low chord, 500 in the upper chord, 800 in the diagonals, and 1 600 in the verticals. These low values indicate the eminent freedom of the arch from secondary stresses under full or uniform loading.

The second diagram on Plate XLVI is similar in all respects to the second diagram on Plate XLV, and the stresses are found to be proportional. This affords a check on the correctness of the computations, as the respective stresses were found by entirely different and independent procedures.

Plate XLVII presents a compilation of all the secondary stresses and a comparison with the corresponding primary stresses. The data on this plate are self-explanatory.

The secondary stresses of greatest critical interest are those occurring with the loading that produces the maximum primary stress in each member. The total secondary stresses for such condition, including both the dead and live-load contributions, are tabulated in Column 5. These secondary stresses, with a few exceptions in each group, are found to be less than 1 500 for the lower and upper chord members, less than 3 000 for the diagonals, and less than 3 500 for the verticals.

The last column gives the maximum values of the secondary stresses expressed as percentages of the corresponding primary stresses.

It will be observed that most of the percentages, especially those in the chord members, are small.

In the lower chord, the critical secondary stresses range from 1 to 12% of the maximum primary stresses.

In the upper chord, the first three members have comparatively high percentages of secondary stresses (180, 40, and 17%), but this is due to the low primary stresses (3 104, 6 650, and 10 140, respectively) in these members. There is an excess of section which amply provides for these secondary stresses. The totals of secondary plus primary stress in these three members do not reach 12 000 lb. per sq. in.

In the other upper chord members the critical secondary stresses range from 3 to 10% of the primary stresses.

In the web members the critical secondary stresses range from 1 to 65%, but only a few exceed 30 per cent. There is ample margin of cross-section to take care of these stresses.

The web members, as a rule, have larger secondary stresses than the chord members. This may appear surprising, as the web members are longer and narrower than the chord members, and it is generally stated that the secondary stresses in different members vary inversely as their slenderness ratios. The reason is due to the form of the arch; in the three-hinged condition practically all the dead load is carried by the parabolic bottom chord, and the web members receive large secondary stresses from the compression of the lower chord members before they receive any appreciable primary stress.

Conclusions.

These results obtained in the secondary-stress computations bear out the justification of the type of structure, unit stresses, and method of erection adopted for the Hell Gate Arch Bridge.

Some of the larger secondary stresses are found at the crown of the arch, and appear to be caused by the special arrangement of the center panel and the part which it plays in the erection of the structure as a three-hinged arch. There were sufficient advantages in this construction, however, to outweigh the increase in the local secondary stresses. Nevertheless, the results confirm the usual objections to intersecting web members. On the other hand, they demonstrate the desirability, from the consideration at least of secondary stresses, of reducing the number of hinges in an arch.

The low secondary stresses in the bottom chords are particularly gratifying, as they set at rest any apprehensions aroused by the great depth of these members.

The results, on the whole, show that special methods of assembling and erection, such as were adopted for the Quebec and Sciotoville Bridges, to counteract the effect of secondary stresses, were not required for the Hell Gate Arch Bridge.

DISCUSSION

H. J. BINGHAM POWELL,* M. AM. SOC. C. E. (by letter).—The writer appreciates the care observed to obtain reliable results in taking the stress measurements on the Hell Gate Arch Bridge, but would question the accuracy of the methods used, that is, to a matter of tenths of a thousandth of an inch claimed. As Inspection Officer in Charge of the Department of Gauges and Standards of the British Ministry of Munitions in the United States, he has had, during the last two years, a wide experience of the difficulties that are encountered in taking fine measurements correct to a tenth of a thousandth of an inch, unless the instruments are extremely accurate and are used in uniform conditions of working, temperature, etc. Mr. Powell.

The principle of measurement of the Howard extensometer appears to the writer to be fundamentally unreliable, because of the following objectionable features: The measuring points of the micrometer caliper are conical, and the zero reading for any given temperature is taken from a "comparison bar" with holes. The member to be measured has, similarly, conical holes for the insertion of the micrometer measuring points. The bearing of the measuring points is not on the tips, but on the sides, and that is where the unreliable feature in the measurement enters. If the micrometer is not held so that the points are "square" to the holes, the bearing on the conical sides is very unequal, and, consequently, the measurement is a false one. Further, it is a most difficult matter to get a satisfactory micrometer "feel" with conical points bearing against the inclined sides of the holes. These sources of error present themselves twice: in taking the "zero" reading on the comparison bar, and again when measuring the member. Thus, the combined error may be considerable. Further, the holes in the member are filled with oil, vaseline, or ivory black paint for protection until used. The filling is removed by a pointed aluminum rod, a small cedarwood stick, and some absorbent cotton. No doubt, every care was taken in removing the filling, but in the confined space and under the generally disadvantageous conditions in which the measurements were taken, it would be difficult to obtain the thorough cleaning out of the holes so necessary for any degree of accuracy. Even with the polished surfaces of screw gauges, the writer has found that a film of oil on the thread upsets the measurements entirely; and the conical holes are not only similar in form to the cross-section of the thread of a gauge, but also give a bearing on a surface instead of a line, as in the former case, and thus aggravate the evil of any film of matter present. Also, the holes in the member are of very imperfect finish of surface, and aid the adhesion of such a film. The only way to remove these factors of inaccuracy is to abolish the use of holes and substitute

* London, England.

Mr. Powell. slightly projecting ball-ended cylindrical studs, which present a smooth surface, easily cleaned, and over which very accurate measurements can be made. These studs can be of very small diameter and only project sufficiently for an ordinary micrometer (a micrometer caliper is unnecessary in this case) to have a bearing on the anvil and spindle. The comparison bar of the micrometer would be a plain rod with the ends ground true, and so the zero reading would be a direct, and consequently, exact, one.

Mr. Waddell. J. A. L. WADDELL,* M. AM. SOC. C. E. (by letter).—This paper is not only of great value on account of the important measurements of both direct and secondary stresses which the author records and discusses, but it is also exceedingly interesting because of the historical treatment of the subject of measuring the actual intensities of working stresses in bridge members.

The practice of stating near the beginning of a paper the history of the subject treated is one that is to be highly commended in many cases, for the reason that it places the reader at once *au fait* with the matter under consideration, and thus arouses his interest and induces him to study seriously what follows.

Reliable information concerning the actual intensities of working stresses in main members of bridges, and especially in their connecting details, is still rather meager, notwithstanding the fact that such information would be of inestimable value to all designers of steel bridges. The difficulty is that the experiments necessary to obtain the required knowledge are expensive, and are generally beyond the reach of the individual engineer. Engineering societies and railroad companies are the logical organizations for conducting such experiments; and, recognizing this, the American Railway Engineering Association (probably the most active and progressive of all our technical societies), some years ago combined forces with a number of the prominent railroad companies and, by a well-evolved series of experiments, secured a mass of most valuable information on the subject of impact on bridges and bridge members. A similar series of experiments on actual intensities of secondary stresses in bridge members and their connecting details might well be undertaken by the same combination of forces.

The Hell Gate Arch experiments, incomplete as they are, constitute a fine start on the investigation of stress distribution; but they should be completed, so as to include the effects of live loads. Such a finished series of experiments would extend our knowledge of stress conditions, would determine the efficacy of new structural features, would provide reliable information concerning the effects of various methods of erection, and would develop improvements in both design and construction. Again, in this manner, there could be ascertained

* Kansas City, Mo.

the relative importance of indeterminate stresses, as well as the actual effects of using redundant members. Mr. Waddell.

Although, as just stated, the scope of the Hell Gate Arch investigation was somewhat limited, the results obtained are certainly valuable; and the engineers who evolved and conducted the series of experiments are certainly entitled to the hearty thanks of the entire Engineering Profession.

F. H. FRANKLAND,* ASSOC. M. AM. SOC. C. E.† (by letter).—This subject is of special importance and interest to all those engineers engaged or interested in the design or construction of important steel bridges, and the author deserves high commendation for his thorough, painstaking, and masterly treatment of a subject acknowledged to be of the highest importance in the development of the science of bridge designing. Mr. Frankland

In planning the work it is doubtful whether the methods selected and devised could be well improved upon, having in mind the purpose in view. Although the paper represents a tremendous amount of work in the calculation of the secondary stresses in the structure, involving the solution of a large number of simultaneous equations for each condition of loading, and the analytical work required in digesting the results and deducing conclusions, it is remarkable that the work has been presented in such condensed form, and yet is fully adequate for complete understanding. This tendency toward elimination of all extraneous matter in technical papers on special subjects is to be highly commended.

The importance of such stress measurements is so great that it might be safely said that such investigations, in the future, will prove one of the most useful aids in the design of bridges of unprecedented span. The writer is of the opinion that the desirability of more and extended investigations of this nature is clearly indicated; and he begs respectfully to suggest that the Society take under consideration the idea of similar stress measurement investigations on the Quebec and Sciotoville Bridges, and the completion of the measurements on the Hell Gate Bridge (those applying to live load), thus securing exceedingly valuable and complete data on the largest bridges, of the cantilever, continuous truss, and arch types, in existence. Owing to the probable great expense of these proposed investigations, which would be too onerous for any one individual to bear, and to the undoubtedly great value to the Profession, it would appear quite fitting and proper for the American Society of Civil Engineers to undertake this work, under the supervision of a special committee. It may be well to mention, in elaboration of the foregoing suggestion, that the determination of the live-load stresses of the Hell Gate

* Kansas City, Mo.

† Now M. Am. Soc. C. E.

Mr. Frankland.

Bridge would complete the measurements for that structure, and that initial stress measurements have already been made for the Sciotoville Bridge, under the direction of Gustav Lindenthal, M. Am. Soc. C. E. The results deduced from stress measurements of three types of long-span bridges would be more valuable and conclusive than those for a single structure.

Mr. Waterbury.

L. A. WATERBURY,* M. AM. SOC. C. E. (by letter).—On page 1059 of Mr. Steinman's paper, the following paragraph, which forms the topic of this discussion, appears:

"The foregoing discussion of results does not include the final stage (2-*H*). In that stage, in the members near the ends of the span, the upper end in every case was found to present a larger average stress than did the lower end. The differences ranged from 450 to 2 800 lb. per sq. in. This anomalous result appears to arise from some unexplained disturbance of stress distribution, and does not represent any error of observation."

This interesting condition of observed stress may possibly be due to an irregular distribution of the stresses at the observed sections, which might be due to the order in which the drift-pins and bolts were replaced by rivets. The great size of the members and the great difference between the stress at the center of member 0-2 from that at the top and bottom, for each section, for Stage (2-*H*), suggest that there might easily be considerable irregularity in the distribution of stresses at each section, between the lines of gauged deformations.

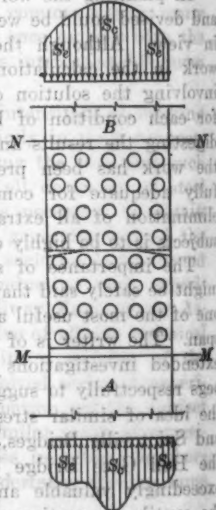


FIG. 7.

To illustrate the principle involved, consider the simple case indicated by the joint shown in Fig. 7, and suppose that the stages of removal of drift-pins and of placing of rivets are as follows: (1) center line of member *A*; (2) outside rows of member *A*; (3) intermediate rows of member *A*; (4) intermediate rows of member *B*; (5) outside rows of member *B*; and (6) center line of member *B*. During each stage a slight closing of the butt joint occurs, which, for joints of the type used in the Hell Gate Arch, develops larger stresses in the middle third of each web than in the outer thirds. At the end of Stage (1), the rivets in place are without shearing stress, but during Stage (2) a slight closing of the joint occurs, producing some stress in the rivets placed during the first stage.

* Nitro, W. Va.

During Stage (3), the rivets placed in the second stage receive some stress, and those placed in the first stage receive an additional increment of stress, but the two intermediate rows of member A are without stress. During Stage (4) a portion of the load which had been carried by the splice plates, is transferred to the web as the butt joint closes, thus relieving the center and outside rows of member A of some stress, but, at the same time, developing some negative shear in the rivets of the two intermediate rows of that member. During Stages (5) and (6), the action on the rivets of member A is similar in character to that which occurred during Stage (4). At the end of Stage (6), the center row of rivets of member B is without stress, the outside rows of the same member have some stress, and the two intermediate rows have considerable stress. Such a distribution of the load among the rivets of the splice would tend to produce a variation of the stress at Section M-M of member A of the type indicated by the stress profile, which is shown below the joint in Fig. 7, while the variation at Section N-N of member B would be more nearly like that indicated by the stress profile shown above the joint. If the deformations are measured along lines coresponding to the stresses, S_c and S_e, of the stress profiles, it is evident that the total stress of member A may be equal to the total stress of member B, in spite of the fact that all the observed stresses for the first member are greater than those for member B. The deformations for member 0-2 for the 2-H stage of erection, as stated in Table 2 of the paper, exhibit this character of variation, notwithstanding the fact that the observed values are large enough to be considered fairly reliable. Mr. Waterbury.

In attempting to check the average stress at sections where there is an irregularity in the distribution of the load, it should be remembered that, even if readings are observed at enough lines to determine the correct variation in deformation for the section, the average stress will not necessarily be indicated directly by the stress profile, due to the variation in thickness of metal across the section. The correct average stress would be obtained by weighting the stress for an increment length of profile, the weight being proportional to the area of metal over which that particular stress is effective, and by computing the weighted mean of all the stresses shown by the stress profile.

F. D. HUGHES,* M. AM. SOC. C. E. (by letter).—Engineering has been correctly called the creative profession, and yet the engineer, who is most directly connected with production of the world's wealth, such as manufacturing and kindred lines, has very little time to devote to independent investigation of engineering problems, and, for that reason, he, more than any other, owes thanks to the authors of this and similar papers who spend their time in investigation on the frontier Mr. Hughes.

* St. Louis, Mo.

Mr. Hughes. of engineering thought and experimental research. The average Board of Directors of corporations looks with scant favor on expenditure of either time or money in search of information and formulas in engineering that do not promise an adequate financial return to the corporation which it represents. Therefore, as already stated, this large body of the Engineering Profession, which, for want of a better term, the writer would designate as industrial engineers, owes much to those who take the time and opportunity to investigate, for the benefit of the Profession at large, such problems as were presented in the construction of the Hell Gate Arch Bridge and to give the results to the engineering world. It is permitted to few engineers to be connected with either the design or construction of a structure of this magnitude, and perhaps the conclusions to be reached are not of practical use to a majority of the Profession, and yet the paper, in itself, is a very valuable contribution to engineering literature.

In an experience of some twenty years, mostly devoted to the design and construction of highway bridges, the writer has found a wide diversity of treatment in the consideration of secondary stresses. Such treatment has varied from that degree of refinement which might be termed "painful" in its exactitude, to that in which the stresses were completely ignored. As an instance, he recalls the construction of a 140-ft. span, designed to carry a concrete floor of the Warren subdivided panel type, all members being riveted, in which the wind stress was carried into the lower chord and exact consideration was given to the increment of stress, even to the changing of the size of section in adjoining panels where the live- and dead-load stresses were of equal moment, thus necessitating (if the theory of the design is followed), a field connection between the two panels where practice should have made the two in one single member. In view of the fact that the stringers were rigidly connected to the floor-beam, the writer considered this degree of refinement a waste of material. On the other hand, he has seen numerous structures of highway design, of the pin-connected type, in which the entire lower chord was composed of eye-bars, and in which the first vertical member consisted of two light angles barely sufficient to carry the computed live- and dead-load stresses; and the detail of connecting the floor-beam below the chord, would, by the same analysis of wind stress, cause bending in the vertical member of from 40 000 to 50 000 lb. per sq. in.; and yet these bridges are standing and have been carrying ordinary traffic for years. Between these two extremes lies the most acceptable practice, and the author's conclusion, that any treatment of secondary stresses should be carefully analyzed and the experiments along the line of stress measurements taken with considerable allowance, is very timely.

The lesson to be learned from the result of these measurements, it seems to the writer, is, that all structures which are designed to

carry their own weight during erection, as well as eccentric loading from machinery or other methods of handling, should have some scheme of measurements for checking the secondary stresses. The additional expense would be more than made up by the insured safety of the structure against probable loss of life and limb, and would have prevented numerous disasters which have occurred on structures of this class. Mr. Hughes.

A. H. FULLER,* M. AM. SOC. C. E. (by letter).—The Engineering Profession is certainly indebted to Mr. Lindenthal for developing the erection of the Hell Gate Arch Bridge into a huge, scientific experiment, and to the author for the admirable manner in which this has been carried out and presented. No one who has given any attention to stress measurements can fail to appreciate the precautions taken in this work and the care in carrying them out, and neither can any one who has ever attempted to compute secondary stresses fail to recognize the magnitude of the task along this line. Each of these features could well be considered an accomplishment, and the combination of the two, on a structure of the magnitude and interest of the one under consideration, marks a new achievement in the understanding of the actual behavior of steel structures. Mr. Fuller.

Engineers would not be doing their duty by simply applauding what has been done without making an effort to analyze the results, in a manner that would throw even greater light on the subject, and to remove the last shadow of doubt in regard to the interpretation that may be drawn.

On page 1065, it is stated that the following relations are evident:

"1.—Except for the smaller percentage of secondary stresses, the measured values are consistently lower than the calculated values.

"2.—As the percentage of calculated stresses increases, the percentage of measured values also increases, but not as rapidly.

"3.—For the highest percentages of secondary stresses, the measured values amount to only a small fraction of the calculated values."

On page 1071, in Paragraph 5 of the Summary of Conclusions, the same points are brought out. A probable interpretation from these conclusions is that measured secondary stresses are usually lower than computed ones.

An examination of Plate XLIV discloses the fact that the recorded measured secondary stresses are as great as, or greater (frequently much greater) than, the computed ones in all cases where the primary unit stresses exceed 5 000 lb. per sq. in. and, with one exception, for all cases where the primary unit stresses exceed 3 000 lb. per sq. in., that is, that the measured stress is greater than the computed one whenever the primary stress approaches a maximum working value.

* Easton, Pa.

Mr. Fuller.

Although the author withholds the measured secondary stresses for the stage, 3-*H,* because of the inconsistencies due to added dead load during observation, the primary stresses are given. It seems improbable to the writer that this disturbance could be so great as to render the results valueless, and he would like to ask the author for these results in connection with the points that have just been raised.

On Plate XLVII are given the percentages of computed secondary stresses in all members, and it is of interest to note, as the author has so well brought out, that these are remarkably low in the lower chord members which carry the greater portion of the load. Take Member 4-6, for instance, in which the computed secondary stress due to dead load is 5% of the primary stress. Plate XLIV indicates that the measured secondary stress is 13% in one truss and 11% in another, of the primary stresses, that is, more than double the computed ones. The writer would like to ask what information is available concerning the variation of stresses within the 20-in. gauge line that was used. The intensity is certainly greater toward the end of the member. Measurements that have been taken by the writer on steel and concrete in reinforced concrete structures indicate a very rapid drop in maximum stresses from the point of connection of the member, and suggest that gauge lengths even less than the usual 8 in. are needed to catch the maximum stresses. This point has been brought out much more clearly by Professor McMillan, of the University of Minnesota, in comparisons between 2-in. and 8-in. gauge lengths. It seems, therefore, that the intensity of secondary stresses may be somewhat greater than the averages over 20-in. gauge lengths, and that they are intensified again at the point of decrease in section at the edge of the gusset-plates.

These points can be recognized without questioning the author's general conclusions that the secondary stresses in the structure under consideration are not large enough to cause concern, and that the type of structure is well chosen to keep them within low values. It does not seem to the writer that it is well to minimize these stresses, for that would tend to give unwarranted confidence and possibly result in overlooking secondary stresses and stress measurements in cases where they might otherwise be secured and thus contribute to the store of knowledge.

The writer would like to ask for more information than has been given in the paper, concerning the distribution of stresses in different portions of the members. Easy computations from the data in Table 2 show the average intensity of the stress in the central angles to be about 50% greater than the average intensity at the four extreme angles for the 2-*H,* or last, stage, for the upper end of Member 0-2.

Similar conclusions may be drawn from Table 4 for the upper end of Member 4-6. The author has well pointed out that the nature of the connections tends to concentrate the stress in the middle portion of the member, but presents only average values, except for the data in Tables 1 to 4. It would be interesting to have any data that may be available for particular locations in regard to the distributions of stress between plates and angles. The writer would not anticipate any great change in work so carefully designed and constructed as this has been, but feels that the information would be desirable. Mr. Fuller.

On page 1051, the author states that the Howard strain gauge is probably the simplest and most accurate field instrument for measuring strains. As the writer has never used this instrument, he is not in the position to question the statement. He is under the impression, however, that more information may be secured from the quicker acting Berry strain gauge for the same expenditure of time and energy. The inference drawn by the writer from the work on the Hell Gate Arch Bridge is that for each load only one measurement was recorded for each gauge length. Possibly this has been verified from several readings and the average recorded. The writer has secured much more consistent results from the Berry instrument by taking three or more sets of readings, with change of dial between each set, and recording all the results, than by taking all the readings at a certain point consecutively and recording an average.

JAMES E. HOWARD,* ESQ. (by letter).—This is a most interesting and instructive paper on the results of measured strains on a structure of magnitude, which, in its conception and execution, occupies the foremost rank in engineering works. The intelligent care which was exercised in acquiring the experimental data, the impartial criticism of this method of measurement, the definition of its proper scope, as well as its limitations, is a source of gratification to the writer, who believes himself to be the one who inaugurated this simple and direct method of examination, a method which is capable of furnishing many useful results which have not yet been placed before the Engineering Profession. Mr. Howard.

This method of measured strains has been used by the writer in the examination of the dead-load strains in bridges of large span, in the structural members of modern buildings, in the distribution of stresses in the sheets and across the seams in steel boilers, in the investigation of the internal strains in steel rails, and also in the study of thermal effects in street pavements. On dates earlier than the examples just mentioned, the method was used in ascertaining the state of internal strains in steel forgings, in railway axles, in cold-rolled shafting, and

* Washington, D. C.

Mr. Howard.

also the residual strains in steel bars after over-straining loads had been applied and released.

This method of examination was first used by the writer, in 1886 and 1887, in the determination of the internal strains in steel forgings, in lieu of a method used by Gen. Nicholas Kalakousky, of the Russian Artillery, who utilized a microscopic cathetometer for the purpose.

The use of drilled and countersunk holes for defining the gauged lengths enabled such lengths to be established and preserved against accidental injury, even against vicissitudes of street traffic, in the examination of street pavements; in structural members, protection is afforded to the contact surfaces during handling and erection, in addition to which the feature of permanence is present, enabling a re-examination of the gauged lengths after the lapse of time.

A reference bar, having substantially the same coefficient of expansion as that of the work under examination, serves its purpose as a standard of length. The reference bar is kept at the same temperature as the work, whenever it is practicable to do so, at other times a correction for the difference in temperature is applied.

Two types of strain gauges—or transfer instruments, as they might be termed—have been used: one is adjustable to any gauged length from 1 in. upward, 36 in. being considered a practical maximum, the other being represented in the type used by the author. The adjustable gauge was the earlier design. It was intended for use on horizontal surfaces or those nearly so, and is more essentially a laboratory instrument than the type used on the Hell Gate Arch Bridge. The latter admits of being used in different positions.

The subject of measured strains is one which is particularly attractive to the writer, as it appears to open the way to advance knowledge relating to the actual conditions or states of strain which pertain to engineering structures of all kinds. Dead-load strains, equivalent to 20 000 lb. per sq. in., have been observed in bridge members. Temperature differences have been responsible for stresses of several thousand pounds per square inch in large members. In steel rails, wheel pressures have caused the introduction of internal stresses, exceeding 20 000 lb. per sq. in., in the head of the rail next the running surface, with tensile components in the interior of the head, stresses which, singularly, appear to have been overlooked in railway engineering.

It is felt that a most promising opportunity for the acquisition of engineering data has been permitted to pass unimproved, or has been utilized to a limited degree only, in the few examples of measured strains in engineering structures. It is with great pleasure, therefore, that the writer witnesses the presentation of this valuable paper which was provided for by Gustav Lindenthal, M. Am. Soc. C. E., the Chief Engineer of the magnificent Hell Gate Arch Bridge.

ISIDORE DELSON,* ASSOC. M. AM. SOC. C. E.—Several years ago, the Bridge Department of New York City completed the work of strengthening the end spans of the Williamsburg Bridge across the East River. In connection with this work, a great many strain observations were made with the Howard extensometer, in co-operation with the U. S. Bureau of Standards. Mr. Delson.

The speaker desires to describe here especially one series of observations which seem to him of particularly striking interest, inasmuch as they actually assisted in the diagnosis of a perplexing situation, and pointed out the safe method of procedure in a difficult operation.

As indicated in Fig. 8, the end span trusses were supported in the lower chord, at Panel Point L 29, by 10-in. pins, by which the reaction was transmitted to the main, that is, suspension, span of the bridge through the panel triangle, L 29-U 30-L 30, containing the tension

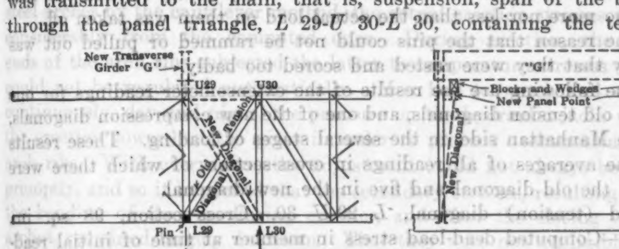


FIG. 8.

diagonal, L 29-U 30. As a part of the reconstruction, it was required to relieve these pins of their loads, to remove the pins, and then replace them by others of greater diameter.

At each truss a new panel triangle (U 29-L 30-U 30), containing the compression diagonal, U 29-L 30, was built out from the main span. The vertical end posts, U 29-L 29, of the end span trusses were connected to a new overhead transverse girder, G, which was supported by cast-steel blocks and wedges on the new panel points.

The relief of the pins was effected by wedging up the girder, G, from the new points of support, and the end spans were thus made to hang from that girder. As the old tension diagonals were being relieved of tension, the new diagonals were being compressed.

Extensometer points had been established on the old tension diagonal, and, also, on the new members before their erection.

It was determined by computation that the girder, G (and the trusses hung from it), would have to be wedged up ¾ in., in order to relieve the pins of the dead load re-action. However, when this wedg-

* Stapleton, N. Y.

Mr. Delson. ing was effected, the pins absolutely refused to budge. The wedging was then increased to 1 in., and still the pins could not be made to move. The contractor apparently saw no other way but to continue this wedging process, and he was permitted to go as high as 1⅜ in., but without success.

All operations were followed up with extensometer observations on the members affected, but particularly on the diagonals. The results of these observations convinced the engineers of the Department that the contractor was on the wrong tack. Accordingly, the wedging-up process was stopped, and the condition was restored to the first established ¾-in. stage. Then the pins were removed by cutting them in halves. On their removal, there was not the slightest sensible dislocation of the members assembled on them, which verified the fact that no more nor less than the actual load on them was taken off.

The reason that the pins could not be rammed or pulled out was simply that they were rusted and scored too badly.

The following are the results of the extensometer readings for one of the old tension diagonals, and one of the new compression diagonals, at the Manhattan side, in the several stages of loading. These results are the averages of all readings in cross-section, of which there were six in the old diagonal and five in the new diagonal.

Old (tension) diagonal, *L* 29-*U* 30.—Cross-section, 98 sq. in., gross.—Computed dead-load stress in member at time of initial readings, 11 500 lb. per sq. in.

Apparent relief of stress.—Averages of six readings in cross-section.

Stage of wedging..	¾ in.	1 in.	1⅜ in.	¾ in.	After removal of pin.
"Relief", in pounds per square inch..	11 300	13 600	16 600	11 500	11 600.

New (compression) diagonal, *U* 29-*L* 30.—Cross-section, 148 sq. in.—Computed stress for dead-load reaction, 6 400 lb. per sq. in.

Apparent stresses.—Averages of five readings in cross-section.

Stage of wedging..	¾ in.	1 in.	1⅜ in.	¾ in.	After removal of pin.
Compression, in pounds per square inch.	6 500	7 700	10 500	6 650	6 300.

Attention is called, not only to the close agreement of the computed stresses with those disclosed in the ¾-in. stage, but also to the fact that, after the pins were removed, there was no change in the stresses of the members, showing that just the exact amount of wedging had been applied.

It should be noted that the series here presented is rather exceptional, and that such good results may not always be expected from the extensometer; but, even where results are not so satisfactory, they, as a rule, would be close enough to help greatly in arriving at a correct judgment of conditions for all practical purposes. Mr. Delson.

GUSTAV LINDENTHAL,* M. AM. SOC. C. E.—A few words may be added to this able paper as to the speaker's reasons for having the erection stresses measured, as related by the author. He wanted to ascertain what, if any, bending stresses remained in the trusses after erection. Mr. Lindenthal.

The speaker had in mind that, during the erection of the St. Louis Arch Bridge (built by the late J. B. Eads and Henry Flad, Members, Am. Soc. C. E., in 1870-74), which was the first instance of erecting steel arches by the cantilever method, the actual erection stresses varied considerably from the computed ones. During erection, the fixed ends of the arch ribs subjected the latter to temperature stresses which could not be regulated automatically. Men had to watch and adjust continuously, day and night, the pressure on the hydraulic jacks under the erection towers on the piers in order to prevent overstressing the arch ribs. With every care, the adjustment could not always be done promptly, and so it happened that the secondary stresses, resulting from the bending of the ribs during erection and after the closing of the arches, remained largely unknown. That they were not negligible was shown by the fact that after erection one of the steel tubes forming the chords broke at the joint (and was replaced).

In addition to the bending stresses from erection and temperature changes are the secondary stresses from live load. These stresses must also be quite large in the St. Louis Bridge from the alternating flattening and bulging of the ribs at the quarters. An incidental effect of this is the creeping of the rails (each track is carried by two ribs) in the direction of the traffic to the extent of 12 in. per day. Another is the excessive straining of the cross-diagonals between the ribs, when one pair of ribs flattens and the other pair bulges under meeting trains. That this bridge has stood these extraordinary strains thus far without failure is proof of the excellent quality and elasticity of the crucible steel in the chords. If the strains in them could be measured (which they cannot, because the steel staves are enclosed in an envelope of wrought iron, and are not "get-at-able"), it would be a most instructive guide in other arch designs.

In the design for the Hell Gate Arch, the speaker desired to minimize as much as possible the secondary stresses (from change of form). One way of doing this was by having the trusses stiff.

By making the arch deep at the quarters and three-hinged for dead load (which in this case is 40% greater than the live load) and two-

* New York City.

Mr. Lindenthal.

hinged for live load, it could be arranged to avoid reversal stresses in the bottom chord altogether, and to keep the range (about 8 000 lb.) of unit stresses from minimum to maximum compression to within one-fourth of the elastic limit for compression on the gross net section.

A further means of minimizing the secondary stresses in the bottom chord was by providing against excessive edge pressures at the joints, as described by the author and also by Mr. Ammann.

In this respect, the Hell Gate Bridge is the antithesis of the St. Louis Arch Bridge with its tube sections and sleeve connections which are more subject to large edge pressures than, for instance, the full contact riveted splices in the shallow ribs of the Clifton Arch Bridge at Niagara Falls, which was likewise erected by cantilevering.

The object of the strain measurements, as related by the author, was fully attained. There are no unknown stresses in the Hell Gate Arch structure.

Mr. Parcel.

JOHN I. PARCEL,* ESQ. (by letter).—The writer has read this very able paper with great interest. The introduction of the strain gauge marks a definite epoch in the development of the science of structural design, and the remarkable large-scale experiment which this paper reports can hardly fail to encourage a much more extensive application of this instrument to the practical problems of the structural engineer.

Certain of the author's conclusions on secondary stresses seem to the writer to be open to argument, and he would like to raise a few questions regarding them. As bearing on secondary stresses in general, perhaps the two most important conclusions stated in the paper are:

1.—Actual secondary stresses, in general, will be less than the computed stresses, due largely to a constant automatic re-adjustment of the internal strains tending toward the relief of secondary stresses.

2.—For most bridges it will probably be satisfactory to provide for secondary stresses by an allowance in the specified unit stresses of about 20% of the total primary stress, plus 3 000 lb.

Perhaps the first conclusion is entirely sound, but some question may be raised as to its deduction from the data of this experiment. Fig. 5, as the author explains on page 1066, shows the curve of measured secondary stress approaching the calculated curve continuously from Stage 1 to Stage 6, as the joint condition approaches more nearly the rigidity assumed in the computation of secondary stresses, diverging again as a release of direct compression permits partial hinge action at the joints, and finally crossing above the calculated curve after Stage 12, when the compression is so great that the joints are completely fixed. At the final stage, 2-H, when the primary stress

* Associate Professor, Structural Eng., Univ. of Minnesota, Minneapolis, Minn.

reaches its maximum, the average measured secondary stress is more than twice the computed value. The action of the three-faced butt joints, in concentrating the direct stress near the middle of the section so as practically to nullify the bending effect, shows a remarkably happy solution of the problem in the chords of the Hell Gate Arch, but it remains true that the measured bending stresses are more than double their calculated value. A. H. Fuller, M. Am. Soc. C. E., has pointed out that, for nearly all cases where the primary stress is greater than 3 000 lb. per sq. in., the measured secondary stresses are greater than the corresponding computed stresses. Plate XLIV shows eighteen measurements for secondary stresses where the measured primary is 3 000 lb. or more, and in all but two cases the measured value is equal to or greater than the calculated value. For the final stage, 2-*H*, the average of the computed secondaries is 6% and the average of the corresponding measured values 13 per cent. That these two figures are fairly comparable is indicated by the author's formula (page 1063) where he takes 12% as an average value for secondary stress proper—*i. e.,* that induced by rigidity of joints when the truss distorts in its own plane. The writer cannot help feeling, with Professor Fuller, that this point is of more significance than the author seems to attach to it, and he would like to inquire if there are reasons, other than appear on the surface, for minimizing its importance. If not, the following additional conclusion might fairly be drawn; "for the higher ranges of primary stress, the actual secondary stresses will probably exceed the calculated values by a considerable margin." As in all ordinary cases it is only the secondary stresses that co-exist with the maximum primary stresses that affect the design of the structure, this result would seem more significant practically than the fact that the higher percentages of calculated secondary stress are not borne out by the test.

Mr. Parcel.

The second conclusion noted above, regarding an allowance in the specified unit stresses of 20% plus 3 000 to cover secondaries, is of interest in its attempt to distinguish between the two kinds of secondary stress: that which is proportional to the primary, and that which is independent of it, a method which seems quite logical and clearly justified by the data of the test. But it may well be asked here whether any empirical relation based on averages is an adequate means of providing for secondary stresses, even in ordinary structures. Perhaps the author does not mean the formula to be used as a substitute for calculations, but he says, on pages 1063 and 1064:

"It is recognized, of course, that a rule of this form can properly apply only to structures in which the secondary stresses have normal or average values. Where unusually large secondaries may be anticipated, these should be given special investigation and attention."

Mr. Parcel. This would seem to imply clearly that only the unusual cases need such investigation. To the best of the writer's knowledge, some form of average allowance applied to all members is the common method of providing for secondary stresses in most structural offices, except for very large structures or those of unusual types. None the less, such a practice appears to the writer to involve a serious inconsistency.

Let it be assumed, first, that the theoretical calculation of secondaries gives results even roughly reliable. Now, it is well known that standard types of simple trusses of moderate span will show effective secondary stresses (those occurring simultaneously or nearly so with maximum primaries) ranging from 5% in some members to 50% or more in others. Take, for example, the 396-ft., pin-connected, Petit truss.* This is fairly typical of the commonest type of trusses for spans ranging from 300 to 600 ft. In four of the seven top-chord members the effective secondary is about 15%; in the others it is more than 60 per cent. To say that the average is 35%, and thus design the structure, would seem quite indefensible, in view of the standard of accuracy maintained in the calculation of primary stresses.

If we assume that theoretical computations for secondaries are wholly untrustworthy, and thus justify reliance on an empirical formula for ordinary bridges, how are we to rely on our analysis for the cases where "exceptionally large secondaries may be anticipated", and what basis is there for the elaborate and very expensive correction methods used in the erection of the Quebec and Sciotoville spans, methods which have been referred to in current technical literature as the most remarkable single feature of these great engineering projects?

We need a great many more experimental data to settle the question of actual *versus* calculated secondary stresses, no doubt; but the bulk of the evidence, up to the present, would seem to justify considerable confidence in our theory. Most previous tests, as shown by the author's citations, show fair agreement between the calculated and observed stresses. In how far the large discrepancies in the test under discussion are due to the unique conditions of the structure is uncertain. The author evidently feels that a considerable part of the discrepancy is due to a relief of the secondaries by re-adjustment of internal strain, and the writer does not care to dispute this point; but the relatively close correspondence between the measured and calculated curves at Stages 5 and 6, and at 3-*H* and 2-*H*, where the actual condition of the joints approaches that assumed in the calculations, seems most significant, and until we have more definite experimental evidence that our theoretical calculations are untrustworthy is it not worth while, even

* As analyzed in the *Bulletin* of the American Railway Engineering Association, January, 1914, p. 454.

in structures of ordinary span, to provide, at least, for secondary stresses on the basis of some approximate analysis? Mr. Parcel.

It is stated on page 1064 that a comparison of individual values of calculated and measured secondary stresses is not very illuminating because of the many disturbing factors which would tend to obscure the systematic variations. The writer has had some part in a recent test to determine some of the secondary stresses in the 518-ft. riveted span of the Norfolk and Western Bridge over the Ohio River at Kenova, W. Va. This test, made possible by the generous co-operation of J. E. Crawford, M. Am. Soc. C. E., Chief Engineer of the Norfolk and Western Railway, was in no wise comparable in magnitude to the Hell Gate Arch test, the principal object being to compare the computed with the measured secondary strains under service loading at a few typical joints. The data of the test are not yet completely

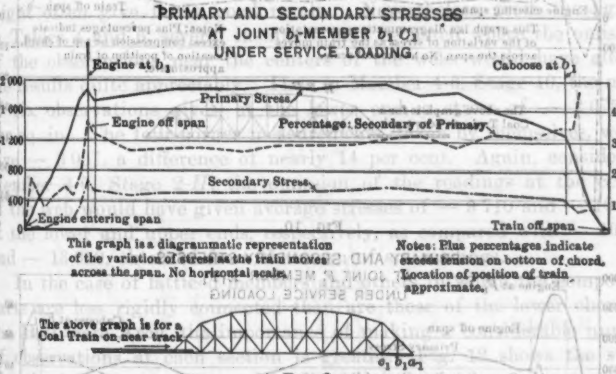


FIG. 9.

analyzed, and any general conclusions deduced would be premature. The graphs of Figs. 9, 10, and 11 are fairly typical, as direct reductions of field data, and the writer has thought they may be of some interest to members of the Society as they stand.

Before making a quantitative comparison with calculated values, the data must be corrected for the location of the instruments, as it was never practically possible to take readings at the extreme edge of the cross-section or the extreme ends of the members. These corrections will generally result in greatly increasing the secondary stress percentages. Before the test was made, the truss was analyzed by joint loads, and influence lines were constructed. These showed that the bending at the lower chord point, b_1, maintained the same direction for all loads, and that the maximum secondary occurred at full loading, though the upper chord points, G and F_1, at bending, reverse during the passing of a single load, and the maximum secondaries occur under

Mr. Parcel. partial loading. This effect is very clearly brought out, qualitatively, in the curve shown. Several complete sets of readings were taken on each joint tested, and the results were so consistent that, when the data are fully analyzed, it is hoped that a joint-by-joint comparison can be made between measured and computed secondaries, which will be of some interest and value to the Profession.

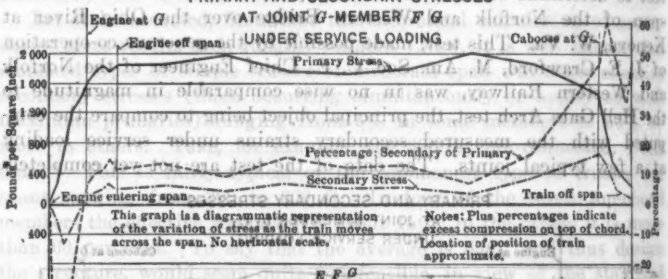


FIG. 10.

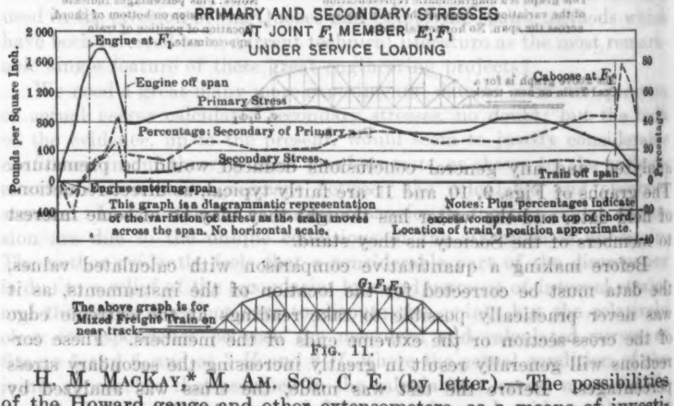


FIG. 11.

Mr. MacKay. H. M. MacKay,* M. Am. Soc. C. E. (by letter).—The possibilities of the Howard gauge and other extensometers, as a means of investigating the distribution of stress in structures, do not seem to be as widely appreciated as they should be. The writer, therefore, welcomes this valuable paper, not only for the information it contains, but because

* Montreal, Que., Canada.

it is likely to attract attention to a method of investigation which deserves to be used far more frequently. Mr. MacKay.

The author states:

"To secure full information as to the distribution of stress in any member, it is necessary to take measurements at not less than three, and preferably four, points of the cross-section near each end of the member."

The writer believes that a considerably greater number of points are usually desirable in large members; and he considers the statement: "The intensity of stress at any other point of the member can then be found by planar and linear interpolation," likely to prove very misleading. If literally accepted, the result would often be to cast doubt on this method of investigation. In the case of box members, to be sure, and other members with rigid transverse webs, such interpolation might often give fairly good results. Nevertheless, from the figures in Tables 4 and 6, several instances may be noted where the omission of the observations near the centers of the webs would have altered the results quite appreciably. Thus in Member 4-6, Stage 10, the mean of six observations gives, at the lower end, a stress of — 2 675 lb. per sq. in. The four corner measurements, taken by themselves, would give — 3 037, a difference of nearly 14 per cent. Again, considering Member 4-6, Stage 2-*H*, the omission of the readings at the center of the web would have given average stresses of — 9 710 and — 10 980, at the lower and upper ends, respectively, as compared with — 10 250 and — 13 050, obtained from the mean of six readings.

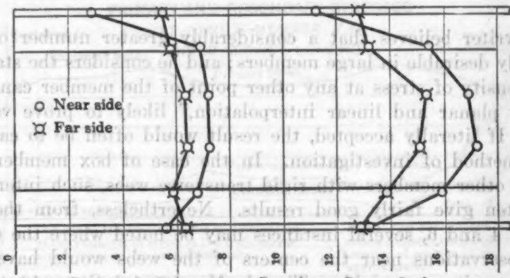
In the case of latticed members and others in which the component parts are less rigidly connected than are those of the lower chord of the Hell Gate Arch, the importance of making a considerable number of observations at each section is greater. Fig. 12 shows the stress distribution, at two stages of loading, in a latticed member tested by the writer, and consisting of four 4 by 4 by ⅝-in. angles and two 22 by ⅝-in. webs. The load was applied through 7-in. pins, and the section represented was near the inner end of the pin-plates. The stresses are plotted on an elevation of the member from a base line at the left of the figure, so as to show the distribution at a glance. Taking E as 29 000 000 lb. per sq. in., we have:

Average measured stress....	— 6 025	— 14 430
Actual stress...............	— 6 380	— 14 720

Although the pin-plates were of ample length and well designed, the concentration of stress near the center of the web is clearly marked. Many such instances might be given, the condition being quite characteristic of the stress distribution near the ends of pin-connected members. It is clear that in such cases planar interpolation from three or four points would give very misleading results.

Mr. MacKay.

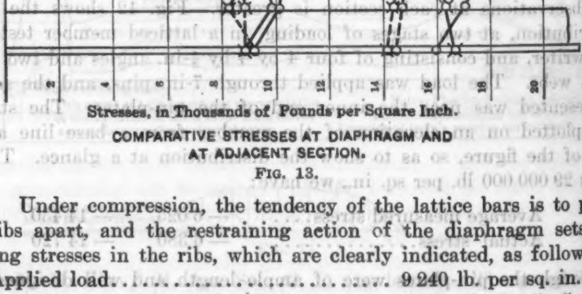
Fig. 13 shows the stress distribution in a similar, but somewhat heavier, member. The full lines indicate the stresses over a 10-in. gauge length, at the center of which a transverse diaphragm is inserted; the dotted lines show the stresses over the same gauge length, the center of which is 10 in. from the diaphragm. Fig. 14 indicates the relative positions of the lattice bars, diaphragm, and points of measure-



Stresses, in Thousands of Pounds per Square Inch.

DISTRIBUTION OF STRESS NEAR PIN-PLATES AT TWO STAGES OF LOADING.

FIG. 12.



Stresses, in Thousands of Pounds per Square Inch.

COMPARATIVE STRESSES AT DIAPHRAGM AND AT ADJACENT SECTION.

FIG. 13.

ment. Under compression, the tendency of the lattice bars is to push the ribs apart, and the restraining action of the diaphragm sets up bending stresses in the ribs, which are clearly indicated, as follows:

Applied load..........................	9 240 lb. per sq. in.
Average measured stress at diaphragm.....	9 920 " " " "
" " " 10 in. away......	9 120 " " " "
Applied load..........................	15 530 " " " "
Average measured stress at diaphragm....	17 630 " " " "
" " " 10 in. away......	15 770 " " " "

All measurements were taken on the outside of the ribs. Although the diaphragm was designed so as to be fairly flexible, it increases the stress in the rib nearly 12 per cent. A shorter gauge length would possibly indicate higher bending stress, and a stiffer diaphragm would undoubtedly increase the effect. Linear interpolation between observations taken at the ends of a member would miss such tertiary effects altogether; and those interested in stress measurements cannot be too often reminded that a thorough study of the action of built-up members demands a complete survey of the whole member. Mr. MacKay.

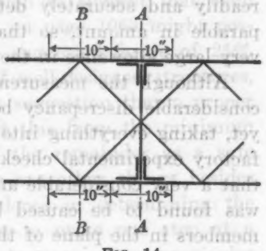


FIG. 14.

The writer cannot share the apprehensions of those who consider the Howard gauge too defective in principle to give good results. It is not easy to protect external holes for a long period on outside work. Any accidental blow in the vicinity of a point of observation may vitiate the results for comparative purposes; also, the writer does not know of any entirely satisfactory material for plugging the holes. Other proposed methods, however, would present similar difficulties. If the gauge is tilted, the contact is between an ellipse and a circle, and, provided that the points are conical, the maximum error is the difference between the semi-major and the semi-minor axes of the ellipse. With a 55° point and a hole 0.03 in. in diameter, a tilt of 5°, which is extravagant, would allow a maximum error of about 0.0001 in.

F. E. TURNEAURE,* M. AM. SOC. C. E. (by letter).—The measurements on the Hell Gate Bridge are of great value, not only in connection with the determination of stresses in that structure, but also as an important contribution to our knowledge of the amount of secondary stress actually existing in structures. The writer does not believe that it will ever be necessary or desirable to calculate secondary stresses in the ordinary practice of bridge design; but he does believe that their consideration, both theoretically and experimentally, in such unusual structures as the one under consideration, is of very great importance. The more accurate our information may be concerning the various kinds of stresses which exist in a structure, the more safely and economically can such a structure be designed; and, although the extra margin of safety required to take care of uncalculated and indeterminate stresses may not be a very serious matter in structures of ordinary size, it becomes of extreme importance in those of very large size. Both safety and economy demand that as complete knowledge as possible should be obtained concerning all the Mr. Turneaure.

* Madison, Wis.

Mr. Turneaure. elements involved. Secondary stress is one of those elements; and, although ordinarily not as large as the primary stress, and not as readily and accurately determined, it is, in many cases, quite comparable in amount, so that it cannot be ignored without making a very large allowance in the margin of safety.

Although the measurements made by Mr. Steinman show a very considerable discrepancy between the calculated and observed values, yet, taking everything into consideration, they constitute a very satisfactory experimental check of calculated results. Nor is it surprising that a very considerable amount of the secondary stress, as measured, was found to be caused by other factors than the bending of the members in the plane of the truss, which is that part of the secondary stress subject to calculation. In tests made by the writer, the same result was obtained in many cases, but where conditions were perfectly symmetrical, the experimental and calculated results agreed very well.

It does not seem to the writer that it is quite safe to draw conclusions relative to secondary stresses in ordinary trusses from the results on the Hell Gate Arch. It is apparent that the arch as designed is in a very favorable condition relative to secondary stresses when under either full dead or full live load. Under such load the main arch rib or lower chord receives its maximum stress, and, therefore, the significant secondary stresses in this member will be those occurring for this condition of loading; but, when the structure is thus loaded, the upper as well as the lower chord is in compression, and the web members are not very highly stressed. This condition results in comparatively low secondary stresses in the chords, much lower relatively than those which occur in an ordinary truss in which the lower chord is in tension and the upper chord is in compression. In the latter case there will evidently be greater distortion of joints than where the chords are subjected to the same kind of stress. As an illustration of a most favorable condition, consider the legs of an elevated water tank If these are made straight and are centrally loaded at each end, there will be practically no secondary stress except for wind pressure. It would appear, therefore, that the secondary stress to be anticipated in the main chord of the arch would be relatively small, as results of measurement as well as calculations show to be the fact. Calculations also show that, for a live load covering a half span, the secondary stresses reach fairly large values, but these are of no particular significance for the main chord, as the primary stresses for this condition are small. It will be noted on Plate XLVI that the calculated secondary stresses for this condition of loading reach a value as high as 2 000 lb. per sq. in., which is an indication of what may readily occur in other types of structures where the maximum primary stress occurs at the same time. It would seem,

therefore, that no general conclusions can be drawn from these calculations and measurements which can be safely applied to other types. It probably was not Mr. Steinman's intention to suggest that such application could be made, but a statement on page 1063 might possibly be interpreted in this direction. His suggested figure of 25% + 4 000 would probably cover most cases of well-designed structures, but the writer would hardly agree with his suggestion that a proper way to treat the matter would be to deduct this from the minimum elastic limit of the material and then use the remainder as a safe working stress for bridge members. There are other things besides secondary stresses which need to be considered in determining the working stresses, and care should be taken to make one step at a time. If it is found that the secondary stresses in any particular structure can be represented safely by a particular percentage of the primary stresses or a percentage plus a constant, then we have arrived at a fair value of the maximum fiber stress, and are much better prepared to discuss the subject of working stresses than if no estimate of the secondary stresses had been made. Mr. Turneaure.

It is evident that secondary stresses are much more serious with respect to compression members than tension members, and the precautions taken in the construction of the Hell Gate Bridge to secure fairly concentric loading of segments should be effective in preventing any very extreme secondary stress. Assuming the pressures along the line of contact of the webs (one-third of their width) to vary as much as from zero at one edge to a maximum at the other, the greatest secondary stress which could be produced under such conditions would be approximately 35 per cent. It was to be expected that the actual stresses would be much less than this.

The great care taken in the design and erection of the Hell Gate Arch and the special precautions observed in other large structures of recent design show the increasing appreciation of the importance of scientific methods of design and accurate fabrication of important structures. Where such conditions prevail, the objection to the use of statically indeterminate structures, which has been so general in the past, loses its force, and there would seem to be no reason for excluding such forms of structure where, for other reasons, they are desirable. These are often called indeterminate structures, but such is not the case. They are simply statically indeterminate, and the calculation of the stresses requires a little more work than for structures that are statically determinate. The results of calculation, also, are not quite so accurate, as they are affected somewhat by temperature variations and inaccuracy of fabrication; but they have exactly the same degree of accuracy as the calculations of deflections of statically determinate structures, and it is well known that deflection calculations are very reliable.

Mr. Turneaure.

The same comments hold true, to a large extent, with reference to secondary stress, and such stresses are just as much a real part of the total stress which may prevail in any fiber of a member as the primary stress; and they are just as certain to exist as it is certain that the structure will deflect under load. The accuracy of their determination, however, is not so great as that of the primary stresses, on account of the indeterminate effect of temperature variations and of joint plates, rivets, and other details.

The very excellent and painstaking work represented by this paper should be an example of what can be done in the scientific treatment of important structures, and the value of the results secured should serve as a strong incentive to other engineers to conduct similar investigations.

Mr. Jacoby.

HENRY S. JACOBY,* ASSOC. AM. SOC. C. E. (by letter).—By having stress measurements made on the Hell Gate Arch Bridge during its erection, an important service has been rendered to the Engineering Profession. The design of bridges, as well as other structures, cannot be carried out with the highest regard for both security and economy without the aid of continuous scientific investigation. When observations are made on an actual structure, especially one in which the members are so large and where the resources of construction in equipment and workmanship are taxed to a much higher degree than usual, the results are of far greater value than laboratory experiments could possibly give.

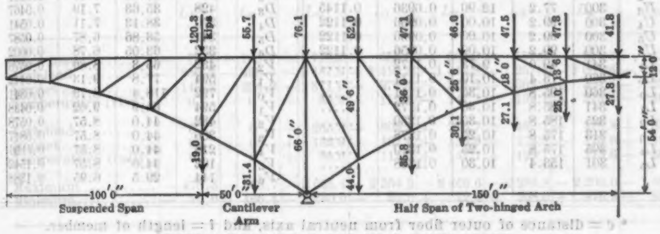
The Chief Engineer, Mr. Lindenthal, therefore merits the appreciation of every engineer who is actively interested in the progress of bridge design and construction, for deciding to take advantage of this unique opportunity. If a larger number of engineers in charge of construction were willing to combine a relatively small amount of scientific research with important works in construction, the rate of engineering progress would be materially increased. The value of the results thus secured, under wise direction, may be far greater than the extra financial outlay involved. The author deserves credit for the systematic form in which the paper is presented. It is gratifying to learn that the actual secondary stresses were found to be lower than the computed values.

As a part of this discussion it may be interesting to present the computed secondary stresses in a two-hinged, spandrel-braced arch with cantilever arms. The design of the arch and of the adjacent suspended spans, a critical discussion of various methods for determining the primary stresses, and the computation of the secondary stresses, were presented to the Faculty of the Graduate School of Cornell Uni-

* Ithaca, N. Y.

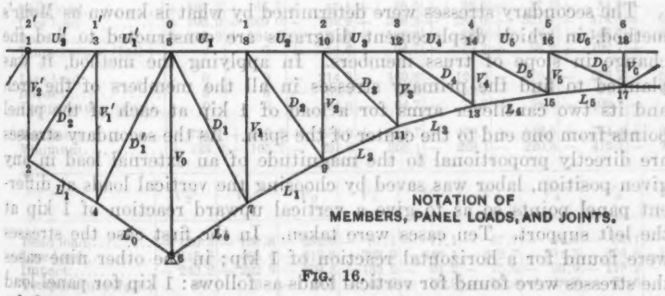
versity as a thesis for the degree of M. C. E. by Mr. Thomson Mao in June, 1917. Mr. Jacoby.

The general dimensions of the trusses, and the dead panel loads, expressed in kips or units of 1 000 lb., are shown in Fig. 15, and the notation used for the truss members, the live panel loads, and the joints, is shown in Fig. 16. The live panel loads on the right half of the arch are 7, 8, 9, 10, 11, and 12; those on the right cantilever are 11′ and 10′. Various properties of the truss members required in the computations are given in Table 7. The revised primary stresses in the members of the half arch and a cantilever arm are given in Table 8. The live panel load per truss is 141.6 kips, the panel load 2′ at the end



GENERAL DIMENSIONS, AND DEAD PANEL LOADS.

FIG. 15.



NOTATION OF MEMBERS, PANEL LOADS, AND JOINTS.

FIG. 16.

of the cantilever being 445.8 kips, of which 375.0 kips equals the reaction of the suspended span. The impact was computed by the formula, $I = \frac{300\,S}{(300 + L)}$, in which I is the impact stress, S is the live-load stress, and L is the loaded length producing the greatest live-load stress. The stresses due to temperature were computed for a range of ± 75° from the mean. The wind stresses were found to be less than 25% of the other stresses, except in the two upper chord members of the cantilever arm. These two members have a large excess of section, as the section of the upper chord is the same for five panels.

Mr. Jacoby.

TABLE 7.

Truss members.	Length, in inches.	Section area, in square inches.	Least radius of gyration, in inches.	Values of $\frac{2c^*}{l}$.	Values of $\frac{2c}{l}$.	Truss members.	Length, in inches.	Section area, in square inches.	Least radius of gyration, in inches.	Values of $\frac{2c}{l}$.
U_2'	300	77.2	12.90	0.0936†	0.1145‡	D_2'	526	51.37	6.95	0.0381
U_1'	300	77.2	12.90	0.0936	0.1145	D_1'	665	51.37	6.95	0.0301
U_1	300	77.2	12.90	0.0936	0.1145	D_1	665	38.13	7.11	0.0301
U_2	300	77.2	12.90	0.0936	0.1145	D_2	526	35.63	7.19	0.0381
U_3	300	77.2	12.90	0.0936	0.1145	D_3	428	35.63	7.19	0.0467
U_4	300	99.2	10.00	0.0956	0.1122	D_4	370	38.13	7.11	0.0541
U_5	300	99.2	10.00	0.0956	0.1122	D_5	341	53.86	6.87	0.0587
U_6	300	99.2	10.00	0.0956	0.1122	D_6	332	63.05	6.78	0.0602
L_1'	341	49.0	9.73	0.1172	......	V_2'	432	65.3	8.80	0.0578
L_0'	360	73.4	10.10	0.1111	......	V_1'	594	77.8	9.13	0.0442
L_0	360	195.8	10.30	0.1111	......	V_0	792	118.4	8.70	0.0341
L_1	341	185.8	10.30	0.1172	......	V_1	594	65.9	9.02	0.0438
L_2	325	185.8	10.30	0.1230	......	V_2	432	44.0	8.57	0.0578
L_3	313	175.8	10.25	0.1278	......	V_3	306	44.0	8.57	0.0817
L_4	305	175.8	10.25	0.1310	......	V_4	216	44.0	8.57	0.1156
L_5	301	153.4	10.30	0.1328	......	V_5	162	44.0	8.57	0.1542
						V_6	144	29.5	6.95	0.1388

* c = distance of outer fiber from neutral axis, and l = length of member.

† Upper side of member.

‡ Lower side of member.

The secondary stresses were determined by what is known as Mohr's method, in which displacement diagrams are constructed to find the change in slope of truss members. In applying the method, it was planned to find the primary stresses in all the members of the arch and its two cantilever arms for a load of 1 kip at each of the panel points from one end to the center of the span. As the secondary stresses are directly proportional to the magnitude of an external load in any given position, labor was saved by choosing the vertical loads at different panel points so as to give a vertical upward reaction of 1 kip at the left support. Ten cases were taken. In the first case the stresses were found for a horizontal reaction of 1 kip; in the other nine cases the stresses were found for vertical loads as follows: 1 kip for panel load No. 0; 2 kips for panel load No. 6; 2.4 kips for No. 7; 3 kips for No. 8; 4 kips for No. 9; 6 kips for No. 10; 12 kips at No. 11; 12 kips upward for No. 11′; and 6 kips upward for No. 10′. The stresses for a panel load of 1 kip at each panel point were found from the foregoing by combining the stresses due to corresponding vertical and horizontal reactions.

After the values of various terms required by Mohr's method for secondary stresses were obtained, the equations were formed for every joint of the truss for each case of loading. Accordingly, it was necessary to solve ten sets of thirty-four simultaneous equations. They

Mr. Jacoby.

TABLE 8.—REVISED PRIMARY STRESSES.

	U_2'	U_1'	U_1	U_2	U_3	U_4	U_5	U_6
Dead load..........	0	+ 70.4	+ 117.3	+ 97.2	+ 73.8	+ 43.8	+ 13.3	0
Live load..........	0	+ 225.3	+ 489.1	+ 582.1	+ 652.4	+ 622.6	+ 406.6	+ 217.9
Impact..............	0	+ 122.7	+ 154.3	+ 183.1	+ 211.1	+ 208.5	+ 143.2	+ 72.6
Temperature (rise)..	0	0	+ 24.6	+ 61.7	+ 117.4	+ 197.2	+ 288.0	+ 332.7
Live load..........	0	0	— 150.3	— 322.8	— 511.6	— 653.4	— 640.4	— 511.2
Impact..............	0	0	— 54.6	— 115.1	— 176.3	— 219.2	— 203.3	— 170.1
Temperature (fall)..	0	0	— 24.6	— 61.7	— 117.4	— 197.2	— 288.0	— 332.7
Maximum..........	0	+ 418.4	+ 785.3	+ 924.1	+ 1 054.7	+ 1 072.1	— 1 118.4	— 1 014.0
Minimum..........	0	+ 70.4	— 112.2	— 401.9	— 731.5	— 1 026.0	+ 851.1	+ 623.2
	L_1'	L_0'	L_0	L_1	L_2	L_3	L_4	L_5
Dead load..........	— 79.9	— 166.1	— 718.2	— 655.5	— 602.5	— 557.6	— 511.3	— 474.7
Live load..........	— 256.2	— 468.4	— 1 487.6	— 1 410.3	— 1 340.8	— 1 257.4	— 1 081.8	— 778.9
Impact..............	— 140.2	— 234.2	— 371.0	— 376.4	— 376.2	— 369.1	— 334.3	— 252.3
Temperature (rise)..	0	0	— 88.6	— 112.1	— 147.1	— 199.7	— 275.6	— 362.1
Live load..........	0	0	+ 265.8	+ 309.2	+ 376.7	+ 452.7	+ 478.8	+ 372.2
Impact..............	0	0	+ 132.9	+ 137.2	+ 150.9	+ 173.2	+ 173.8	+ 127.8
Temperature (fall)..	0	0	+ 88.6	+ 112.1	+ 147.1	+ 199.7	+ 275.6	+ 362.1
Maximum..........	— 476.3	— 868.7	— 2 665.4	— 2 554.3	— 2 466.6	— 2 383.8	— 2 203.0	— 1 868.0
Minimum..........	— 79.9	— 166.1	— 230.9	— 97.0	+ 72.2	+ 268.0	+ 416.9	+ 388.4
	D_2'	D_1'	D_1	D_2	D_3	D_4	D_5	D_6
Dead load..........	+ 123.2	+ 165.7	+ 47.1	+ 35.5	+ 33.5	+ 37.1	+ 34.6	+ 14.9
Live load..........	+ 395.0	+ 369.0	+ 332.9	+ 308.3	+ 280.6	+ 321.1	+ 473.4	+ 654.8
Impact..............	+ 215.9	+ 184.5	+ 120.8	+ 106.2	+ 76.9	+ 86.1	+ 143.7	+ 215.4
Temperature (fall.).	0	0	+ 54.6	+ 64.7	+ 79.7	+ 98.2	+ 102.7	+ 50.2
Live load..........	0	0	— 214.9	— 169.3	— 124.0	— 110.5	— 253.9	— 530.1
Impact..............	0	0	— 69.4	— 54.6	— 52.9	— 48.7	— 94.1	— 179.0
Temperature (rise).	0	0	— 54.6	— 64.7	— 79.7	— 98.2	— 102.7	— 50.2
Maximum..........	+ 734.1	+ 719.2	+ 554.9	+ 514.7	+ 470.7	+ 542.5	+ 754.4	+ 935.3
Minimum..........	+ 123.2	+ 165.7	— 291.8	— 253.1	— 223.1	— 220.3	— 416.1	— 754.4

	V_2'	V_1'	V_0	V_1	V_2	V_3	V_4	V_5	V_6
Dead load..........	— 120.3	— 156.8	— 252.0	— 81.1	— 71.1	— 67.7	— 64.0	— 53.7	— 41.8
Live load..........	— 445.8	— 465.6	— 613.3	— 392.9	— 346.1	— 312.7	— 331.5	— 376.8	— 141.6
Impact..............	— 243.2	— 232.8	— 204.3	— 133.2	— 94.2	— 75.2	— 95.9	— 115.2	— 73.2
Temperature (fall).	0	0	— 48.8	— 53.2	— 56.9	— 57.4	— 48.8	— 21.6	0
Live load..........	0	0	+ 166.5	+ 137.7	+ 82.5	+ 47.9	+ 93.8	+ 189.7	0
Impact..............	0	0	+ 55.4	+ 67.3	+ 40.7	+ 26.1	+ 37.1	+ 69.5	0
Temperature (rise).	0	0	+ 48.8	+ 53.2	+ 56.9	+ 57.4	+ 48.8	+ 21.6	0
Maximum..........	— 809.3	— 855.2	— 1 118.4	— 660.4	— 568.3	— 513.0	— 540.2	— 567.3	— 256.6
Minimum..........	— 120.3	— 156.8	+ 18.7	+ 177.1	+ 109.0	+ 63.7	+ 115.7	+ 227.1	+ 41.8

NOTE.—All stresses are expressed in kips, or units of 1 000 lb.

were solved by the method of Gauss, as they have the same form as the normal equations in geodesy. This fact was first noted by Mr. José Páez, a graduate student at Cornell University in 1912-13, who combined a minor in geodetic engineering with his major subject in bridge

Mr. Jacoby. engineering. Mr. Páez wrote a thesis on secondary stresses in which he made a critical comparison of all known methods for their computation, and applied each of them to the same truss. He also discovered an additional approximate method. By the method of Gauss, the solution of the equations becomes quite mechanical, and may be checked at every step of the process. This method of solution, therefore, has a great advantage over any other.

Mr. Mao made an additional simplification of the solution by numbering the joints in the left half of the truss as shown in Fig. 16, and numbering those in the right half of the truss so that the sum of the numbers for two symmetrical joints is 35, or one more than the total number of joints. By arranging in a table with ϕ_1 to ϕ_{34} at the tops of the columns from left to right, ϕ_{34} to ϕ_1 at the bottom, the numbers of the joints from 1 to 34 on the left side from top to bottom, and the numbers 34 to 1 at the right side, Mr. Mao found that the coefficients were not only symmetrical with respect to a diagonal from the upper left corner to the lower right corner of the table, but also with respect to the horizontal between the numbers 17 and 18. This arrangement saves nearly half the labor by reducing it to 18 equations instead of 34; it requires a table only one-fourth as large as otherwise, so far as coefficients only are concerned; makes a further saving in work in substitutions after some of the values are found, as one substitution gives the coefficients of two unknowns, the sum of the subscripts of which equals 35; it reduces the chance of making errors, as the number of terms to be considered is only half as large as otherwise; gives a more accurate result by avoiding so long a series of substitutions in finding the values of the first few unknowns; and saves time in verifying the check terms.

Fig. 17 gives the secondary stresses in the members of the arch truss, expressed as percentages of the corresponding primary stresses, the loading being such as to cause the maximum and minimum live-load stresses, those for positive primary stresses being shown in the upper part of Fig. 17, and those for negative primary stresses in the lower part. The secondary stresses are for the top and bottom fibers of the upper-chord members, for the top fibers of the diagonals and lower-chord members, and for the outer fibers toward the center of the span in the verticals.

As the members of the cantilever arm have primary stresses due only to one or two live panel loads on the cantilever arm, and the secondary stresses are caused by panel loads over the entire length of the truss, it is necessary to give two sets of percentages for these members. One of these is the percentage of the primary stress for the maximum secondary stress in tension, and the other for that in compression. As U_2' has no primary stress due to vertical loads, the values

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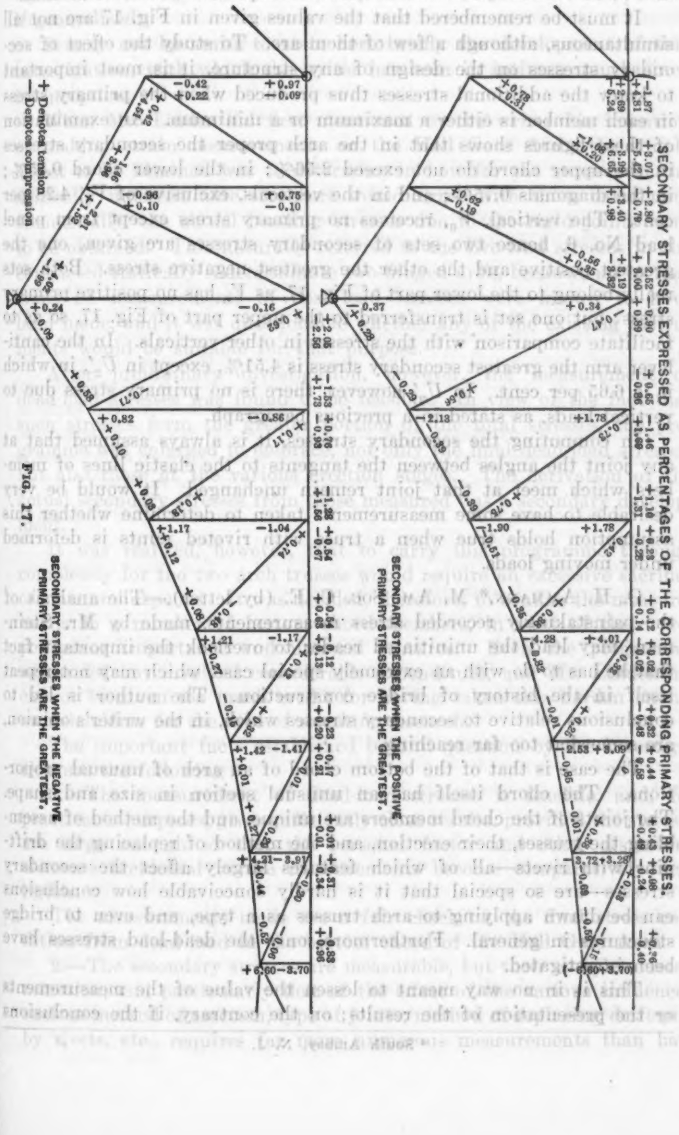

FIG. 17.

Mr. Jacoby. for its secondary stresses are given as percentages of the primary stress in U_1', as both members have the same composition and section area.

It must be remembered that the values given in Fig. 17 are not all simultaneous, although a few of them are. To study the effect of secondary stresses on the design of any structure, it is most important to know the additional stresses thus produced when the primary stress in each member is either a maximum or a minimum. An examination of these figures shows that in the arch proper the secondary stresses in the upper chord do not exceed 2.56%; in the lower chord 0.85%; in the diagonals 0.75%; and in the verticals, exclusive of V_6, 4.28 per cent. The vertical, V_6, receives no primary stress except from panel load No. 6, hence two sets of secondary stresses are given, one the greatest positive and the other the greatest negative stress. Both sets really belong to the lower part of Fig. 17, as V_6 has no positive primary stress; but one set is transferred to the upper part of Fig. 17, so as to facilitate comparison with the stresses in other verticals. In the cantilever arm the greatest secondary stress is 4.51%, except in U_2', in which it is 6.65 per cent. In U_2' however, there is no primary stress due to vertical loads, as stated in a previous paragraph.

In computing the secondary stresses, it is always assumed that at any joint the angles between the tangents to the elastic lines of members which meet at that joint remain unchanged. It would be very desirable to have some measurements taken to determine whether this assumption holds true when a truss with riveted joints is deformed under moving loads.

Mr. Ammann. O. H. AMMANN,* M. AM. SOC. C. E. (by letter).—The analysis of the painstakingly recorded stress measurements, made by Mr. Steinman, may lead the uninitiated reader to overlook the important fact that he has to do with an extremely special case, which may not repeat itself in the history of bridge construction. The author is led to conclusions relative to secondary stresses which, in the writer's opinion, are somewhat too far reaching.

The case is that of the bottom chord of an arch of unusual proportions. The chord itself has an unusual section in size and shape. The joints of the chord members are unique, and the method of assembling the trusses, their erection, and the method of replacing the drift-pins with rivets—all of which features largely affect the secondary stresses—are so special that it is hardly conceivable how conclusions can be drawn applying to arch trusses as a type, and even to bridge structures in general. Furthermore, only the dead-load stresses have been investigated.

This is in no way meant to lessen the value of the measurements or the presentation of the results; on the contrary, if the conclusions

* South Amboy, N. J.

drawn by the author are taken to apply strictly to the case under investigation, or to very similar cases, they are infinitely more valuable than if generalized. Mr. Ammann.

It may not be amiss to state here briefly the development of this investigation, with which the writer has been in intimate touch since its inception.

When Mr. Lindenthal first mentioned to the writer his intention of making strain measurements on the Hell Gate Arch, the simple object to be attained was to determine in how far the stresses in the statically indeterminate structure would agree with those calculated. Incidentally, the actual bending stresses due to the rigid joints were to be obtained. This naturally referred to the live-load stresses in the completed statically indeterminate two-hinged arch, because, up to that time, no measurements of dead-load stresses were known to have been made, and it was questionable whether any of the existing instruments would be suitable for that purpose.

After preliminary investigation, however, the measurement of dead-load stresses was found to be feasible. In view of the fact that such stresses form the greater portion of the total stresses, the programme was enlarged to embrace, not only the final dead-load stresses, but also those at the various erection stages. The derivation of the actual secondary stresses from those measured was a secondary development.

It was realized, however, that to carry this programme through completely for the two arch trusses would require an excessive sacrifice of time and expense. It was decided, therefore, to confine the measurements to a number of bottom chord members, in view of their preponderant importance and unusual features. Even the programme as carried out involved considerable expenditure, and Mr. Lindenthal cannot be given too much credit for having taken this burden on his own shoulders for the sake of scientific research.

The important facts established beyond question by the investigation are the following:

1.—The measurement of dead-load stresses is feasible, and, for practicable purposes, gives sufficiently close results, provided the stresses are large enough so that the personal factor and other disturbing elements incidental to the measurements become comparatively negligible.

The measurements made are a fair check on the final stress condition from dead load in the bottom chord of the Hell Gate Arch.

2.—The secondary stresses are measurable, but their complete determination, and particularly that of the effect of the various influences, such as method of erection, type of joints, method of replacing drift-pins by rivets, etc., requires far more numerous measurements than have

Mr. Ammann.

been feasible without undue cost in the case of such a large structure as the Hell Gate Bridge. For this reason only hypothetical conclusions can be drawn from these measurements as to the magnitude of secondary stresses in arch trusses and bridge structures in general.

The measurements have corroborated the expectation that the secondary stresses in the bottom chords of the Hell Gate Bridge are negligible, that is, are more than covered by the margin of safety of the primary stresses.

3.—The measurements have proved the expected favorable action of the three-face joints of the bottom chord, with regard to avoiding dangerous edge pressures and reducing the secondary stresses.

One important object has not been accomplished, namely, the determination of the actual stresses in the statically indeterminate structure. The dead-load stresses are statically determinate—at least, very nearly so—because such stresses, superimposed after the trusses were converted from two-hinged to three-hinged arches, are very small. In the writer's opinion, it is highly desirable that the measurements be continued so as to embrace the statically indeterminate live-load stresses. They would furnish valuable additional information. As the expense for such further investigation is too heavy for an individual engineer, it should be carried out either by the United States Bureau of Standards or by an Engineering Society in co-operation with the railroad.

Mr. Randorf.

C. A. RANDORF,* M. AM. SOC. C. E. (by letter).—Few engineers have had the experience or the opportunity of conducting tests of the importance and magnitude of those described in this paper; therefore, few can speak with authority on the subject.

In reviewing the results obtained by the author, the writer has been impressed with regard to two points which seem to be most important, namely, the accuracy of the measuring instrument, and the accuracy or rather inaccuracies due to the possible varying ductility of the steel in the bridge.

In the writer's opinion, a micrometer such as the Howard strain gauge is well adapted to work of this kind, due mainly to the simplicity of the instrument, which minimizes the effect of the personal equation. The writer realizes that the observer must exercise extreme care in taking the readings, and must have had experience in operating the instrument, but, with well-trained men as observers, the probable error due to the instrument and its operation, will be less than the actual fluctuating results due to the varying qualities in the steel.

The stress measurements, however, are based on the assumption that the modulus of elasticity for the steel used in this bridge is 30 000 000; undoubtedly, this value is more or less variable, although it is accepted as a constant term in formulas where the elasticity of

* Buffalo, N. Y.

the material is concerned; but, when stresses are computed from actual strain measurements, the modulus of elasticity of the material should be verified by special tests, if possible, in order to ascertain its extreme variations. It is evident that this cannot be done after the structure to be tested has been erected; but it can be done when the regular tests are being made in the laboratory. Mr. Randorf.

The writer refers to the paper by O. H. Ammann, M. Am. Soc. C. E., giving an account of the design and construction of Hell Gate Bridge,* in which is given a résumé of the chemical analysis of the steel used, indicating a variation in the carbon of from 0.27 to 0.34 per cent. This would have a direct bearing on the ratio of the unit elongation or strain to the unit stress, in other words, the modulus of elasticity.

A certain amount of variation or segregation of the carbon will occur in the best commercial steels. Manufacturers have adopted methods to eliminate as far as possible the unequal distribution of carbon and other impurities, but the art of making steel has not as yet become perfect.

With these facts in mind, however, it would seem reasonable to assume that the author did arrive at quite accurate results, inasmuch as the data are based on the averages of a great many extensometer readings.

The writer believes that the data offered in this paper are of inestimable value to the Engineering Profession and well worth careful study by all engineers interested in bridge construction.

DAVID A. MOLITOR,† M. AM. SOC. C. E. (by letter).—The results of stress measurements presented in this paper certainly merit the highest appreciation from the Engineering Profession. Special thanks are due to Gustav Lindenthal, M. Am. Soc. C. E., for his keen forethought and insistence on proving the correctness of the theoretic stress computations in this monumental structure. Mr. Molitor.

These stress measurements undoubtedly possess great scientific value, and, from the engineering viewpoint, it would seem that their main purpose should consist in substantiating the correctness of modern methods of stress analysis, regardless of whether these are simple or complex, so long as the results of stress computations can be accepted as sufficiently reliable to warrant their adoption as the basis for safe designs. In the event that stress measurements could be shown to invalidate the results obtained from modern methods of analysis, the dictates of conscience should prompt us to search farther for truths not yet discovered.

The author, on page 1047, states: "The final object of the extensometer measurements was to provide a comparison between the calculated

* *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 852.
† Detroit, Mich.

Mr. Molitor.

and the actual secondary stresses in the structure." He then mentions the peculiarities in the design and methods of erection and riveting, concluding with the statement: "These features, separately and combined, modify the secondary stresses materially, and render it extremely difficult, if not impossible, to arrive at the true secondary stresses by calculation".

The writer did not find any reference in the paper which would explain how any of these features, especially the three-faced butt-joints, were considered in the secondary stress computations; yet on this point hinges the whole question of how close an agreement could be expected between actual and computed secondary stresses.

These features, as described by the author, and applied to a bridge of this type (two- or three-hinged braced arch), would necessarily result in very small secondary stresses. Therefore, it is most gratifying to know that the good judgment of the designer was verified by the stress measurements which gave secondary stresses averaging about two-thirds of those computed. However, the 50% excess of the computed over the measured secondary stresses may have been due to the manner of computation, if the three-faced butt-joints were not properly considered.

In order to prove or disprove the correctness of the more accurate methods of secondary stress computations, a more suitable structure would have been a simple truss with subdivided panels, wherein the secondary stresses usually attain high values. If the computed values were found to be in close agreement with the measured stresses for such a structure, a fact which is only partly substantiated at present, it would be safe to conclude that the methods are reliable for bridges of all types. To draw such a conclusion from the facts presented in the paper, however, is unwarranted, and is apt to minimize the importance of the question of secondary stresses in ordinary bridges.

The author, on page 1063, in attempting to establish conclusions for guidance in specifying extreme working stresses for bridge members, offers a suggestion which no doubt would meet with great favor among those who are willing to take a chance on the "factor of ignorance" rather than familiarize themselves with modern methods, which are often avoided on the pretext that they are too complicated and laborious. The author states "that a rule of this form can properly apply only to structures in which the secondary stresses have normal or average values", but fails to give any criterion by which to recognize such structures; neither does he convey any good idea of what constitutes a normal value.

The writer's practice is to ascertain the maximum total stress due to dead-live-impact loads, plus secondary stress and temperature effect,

if any, and then design the member for the allowable unit stress. By making proper allowances for live-load stress reversals, he aims to produce a design of uniform strength throughout, for 100% over-load in live plus impact loads, without exceeding the elastic limit. Mr. Molitor.

The ordinary method of analysis, used by the author, involving the solution of a series of simultaneous equations equal in number to the total number of panel points in the span, for each case of loading, is admittedly laborious, and might have been somewhat simplified.

By referring to any treatise on secondary stresses, it is apparent that the theoretic analysis of the problem is quite simple, although the numerical work is laborious, on account of the solution of the final set of simultaneous equations, and, as stated by the author, "the expediting of the entire computation depends largely on the method selected for solving these equations". The method of Gauss is often used to obtain a direct algebraic solution, though many simplifications have been suggested to minimize the labor involved by this method. The method of successive approximations, used by the author, has some merit, though much labor is still required, especially when the process is carried to the third approximate values.

In view of these facts, it would appear that a method of computing secondary stresses which avoids the laborious solution of a set of simultaneous equations would commend itself to bridge specialists and students of this subject.

Such a method has been given, in the chapter on secondary stresses, in a treatise* by the writer, but seems to have entirely escaped the attention of the author as well as others who have dealt with this subject.

It would be transgressing the rules of the Society to quote this analysis here, and as it is treated very exhaustively and illustrated by a complete problem, the reader is referred to the original text. A very much simplified method of solving symmetric equations—known in mathematics as equations of Clapeyron—is also given on page 294 of this treatise, where the solution was effected with a 10-in. slide-rule. This method was also applied by the writer in solving normal equations, up to fourteen in number, and with the same slide-rule, as early as 1907; hence, the discovery by Mr. José Páez, announced by Professor Jacoby in his discussion of this paper, was antedated by at least 5 years, and possibly by more than 40 years, as Mohr's work equations, when applied to many redundants, become identical with normal equations.

The agreement between the calculated and measured values of the secondary stresses in the Hell Gate Arch does not appear to be very satisfactory, from a scientific standpoint, though the disparity is not serious, because these stresses are never excessive in a bridge of this

* "Kinetic Theory of Engineering Structures", Chapter XIII, 1911.

Mr. Molitor.

type, especially when extraordinary conditions are introduced for the express purpose of reducing them still further. The author, on page 1065, evaluates the relation between the measured and calculated secondary stresses in the form of an approximate empiric statement according to which the percentage of measured stress equals half the percentage of the calculated stress, plus 15, and adds that the factor, $\frac{1}{2}$, would probably have a higher value in other structures. Accordingly, one or the other value, attributed to the secondary stress in any member, must be radically wrong. That is to say, either the computed secondary stresses are much too high, owing to some error in theory or its application to the three-faced butt-joints, or else the measured stresses are too low, for some reasons not discernible from the data presented.

On Plate XLVII, the extreme fiber distance for the lower chord is given as 63.6 in. for Member 0-2, with diminishing values to 41.9 in. at Point 22. These are in accordance with Fig. 4, without any allowance for the three-faced joints, and, if these distances were used in computing the fiber stresses from the bending moments, Navier's law has been violated, and the resulting stresses may easily be 50% too high for practically all cases of loading, except the maximum dead-live-impact load, for which no stress measurements were made.

On the other hand, the location of the extensometer points adjacent to the ends of a member may, or may not, give maximum fiber stresses; and as there is only one point at each end of each member for which the fiber stress can be a maximum, there is great likelihood of missing this point in locating the points for measurements, and hence the chances for obtaining consistently low measured fiber stresses are very great. For the direct stresses, this danger is not encountered.

The fundamental equations for computing secondary stresses are expressions representing the relations between the unit direct stresses and the angle distortions resulting from these stresses, for every triangle of the frame. The angle distortions are thus derived on the assumption of frictionless pin connections between all members. These angle distortions, thus found, are then assumed to represent the bending effects in the members which are prevented from taking place by virtue of rigid riveted connections at the ends or joints. Hence, as the material, even at the joints, is elastic, and not rigid, some elastic distortion will take place in the apex angles of every triangle, the extent of which will depend on the quantity of metal and riveting used in the connecting plates, and, whatever this may be, its amount will be lost in the production of bending stresses in the members themselves. For very stiff joints, or heavy details, this will be a small quantity, representing an error on the side of safety in computing the secondary stresses. Therefore, the computed secondary stresses must always be

slightly in excess of the actual values, but the error is relatively small and not in a class with the findings recorded in the paper. Mr. Molitor.

The actual maximum bending stress could hardly occur at the extreme end of a member, but would be developed at some distance back, near the edge of the connecting plate, and that point would have to be located very accurately in order that the measured fiber stresses may be accepted as a true measure of the maximum bending stress. This, again, does not apply in the same way to the direct stress.

Hence, it is quite evident that some disparity will always exist between computed and measured secondary stresses, without casting any serious reflection on the accuracy of either value, unless the doubt can be located definitely.

In conclusion, the writer expresses his sincere appreciation of the author's contribution to this subject, and trusts that the remarks presented herewith may be accepted in the sense of constructive criticism.

D. B. STEINMAN,* M. AM. SOC. C. E. (by letter).—The writer wishes to take this opportunity of acknowledging his cordial appreciation of the courtesy and interest of all those who have discussed his paper. The number, range, and significance of the discussions have been greater than were anticipated for a subject in so new a field, and the interest evinced by so many high authorities in the Profession has been gratifying. Mr. Steinman.

In eliciting this series of valued contributions of data and thought to the science of structural stresses, the writer feels that his labors in conducting the investigations under consideration, and in analyzing and reporting them, have found their full justification and reward.

The various discussions will be taken up in the order in which they were received by the writer.

Mr. Powell, in commenting (page 1077) on the precision of the extensometer measurements, reports that he has found it difficult in his experience with gauges to take readings correct to a tenth of a thousandth of an inch.

The writer does not claim that this degree of precision was secured, nor does he wish to minimize the difficulty of even approximating it. Throughout the work, a number of readings were taken for each gauge measurement until the results appeared to be consistent within two or three ten-thousandths of an inch, and then the average of the last three or four readings was recorded. The proof of the accuracy secured by this procedure, using the instrument and method of manipulation described in the paper, does not lie in abstract considerations, but in the actual comparison of measurements at the two ends of

* New York City.

Mr. Steinman.

each member, or of observations made on the same gauge points on different days.

Mr. Powell regards as objectionable features the conical measuring points of the extensometer and the conical holes for the insertion of these points. It is true that these would give rise to error if the instrument were not held square to the holes, but this is a precaution which is easily observed; moreover, as pointed out by Professor MacKay (page 1097), a slight deviation from the perpendicular produces an entirely negligible error.

The question of getting a satisfactory micrometer "feel" with conical points bearing against the inclined sides of the holes, and of the difficulty of securing a thorough cleaning out of the holes, are also raised by Mr. Powell; but these and other difficulties were fully recognized by the writer and his assistants, and necessitated extreme care in every phase of the work. It may be stated that the cleaning of the holes before each measurement usually consumed more time than the actual taking of the readings; but, with conscientious and practised observers, using every possible precaution, the effects of the above-mentioned difficulties were minimized, and reliable measurements were secured.

If any more accurate instrument were known, or any better method of preparing and taking care of the gauge points, the writer would have been glad to adopt it for the Hell Gate Arch observations. The suggestion made by Mr. Powell to use slightly projecting ball-ended cylindrical studs, instead of the conical holes, with an ordinary micrometer replacing the Howard instrument, is certainly worth trying; but it appears doubtful whether it would yield sufficient increase in accuracy to justify the greater labor required in preparing the gauge points. Moreover, as Mr. Howard suggests (page 1086), the use of drilled holes better enables the gauge lengths to be preserved and protected.

The writer is grateful to Dr. Waddell for his kind remarks on the value and importance of the measurements; and heartily endorses his recommendation that the experiments on the Hell Gate Arch should be completed, so as to include the effects of live loads.

Mr. Frankland suggests an interesting programme of stress investigations, to embrace measurements on the Quebec and the Sciotoville Bridges, and the completion of the tests on the Hell Gate Arch. The undertaking of this work by the American Society of Civil Engineers, under the supervision of a special committee, would certainly extend our knowledge of stress conditions, and develop improvements in both design and construction.

Professor Waterbury proposes an interesting theory to account for the anomalous results found by the writer in some of the members of

Mr. Steinman.

the final stage, where the upper end uniformly presented a larger average stress than the lower end. Professor Waterbury's explanation of the possible influence of the order in which the drift-pins may have been replaced by rivets is very lucid, and is one of the theories which the writer had in mind when he wrote:

"This anomalous result appears to arise from some unexplained disturbance of stress distribution, and does not represent any error of observation."

If Professor Waterbury's theory is correct, there still remains to be explained why the effect described was confined to the members near the end of the span, and why the larger resulting stress was invariably found at the upper end in each of these members.

The point brought out by Professor Waterbury, that the correct average stress at a cross-section can be obtained only by weighting each stress reading in proportion to the area of metal it represents, was recognized by the writer in his early studies. At one time, as a result of a comparison of the respective areas represented by the middle and the corner gauge points, the rule was tried of giving the middle readings twice the weight; but this did not yield as consistent results as the method which was finally adopted of simply averaging the six readings at each cross-section.

Mr. Hughes submits a comparison of the two extremes which he has observed in bridge design practice, *viz.*, excessive and sometimes meaningless precision in computation of stresses and in proportioning of members, on the one hand, as contrasted with slipshod and skimpy detailing of members, in violation of the fundamental principles of safety, on the other. No one will take exception to the recommendation that the most acceptable practice lies between these two extremes.

When Mr. Hughes advises the measurement of stresses on all structures in which there may be uncertainty of condition as a result of erection operations or eccentric loading, he is recommending something highly desirable but probably impracticable. The work of taking stress readings is expensive, and requires carefully selected and well-trained observers.

The writer is indebted to Dr. Fuller for his able and comprehensive discussion, and regards the latter's close study of the paper and his careful scrutiny of the results and conclusions as a high compliment. This contribution eminently succeeds in the object suggested in its opening paragraphs as a guiding purpose for such discussions, namely to "throw even greater light on the subject, and to remove the last shadow of doubt in regard to the interpretation that may be drawn."

Dr. Fuller refers to the writer's statement that measured secondary stresses will usually be lower than computed ones. The only exceptions

Mr. Steinman.

to this rule arise when there are special circumstances in the fabrication or erection of the structure which tend to increase the secondary stresses in a manner not provided for in the computations. An example of such augmentative factor, in the case of the Hell Gate Bridge, was the cantilever method of erection; this unavoidably gave rise to large secondary stresses, which persisted after the structure passed from the cantilever to the three-hinged condition. There was no way of computing the effect of this transition in advance of actual test, consequently the secondary stresses in Stage 3-*H* exceeded the calculated values. This fact was frankly displayed in Table 5 and in the graph on Fig. 5, and the explanation was suggested in the text.

Consequently, when Dr. Fuller points out that wherever the primary stresses exceeded 3 000 lb. per sq. in. the measured secondaries were greater than the computed values, he is not upsetting the writer's conclusions in this regard; he is focussing attention on what is merely an example of the qualifying circumstances outlined in the foregoing paragraph; for it happens that all stresses exceeding 3 000 lb. per sq. in. occurred in Stages 3-*H* and 2-*H*; and in these stages the special conditions just described came into play.

In the more common methods of truss erection, there is no abrupt transition from one structural form to another, like that involved in the cantilever erection of the arch; consequently, in such cases, there will be nothing to invalidate the correctness of the calculation of secondary stresses. This may be regarded as the normal or usual condition obtaining; the final stages of the Hell Gate Arch erection represent an unusual or exceptional condition. With these considerations, the writer does not see any reason for renouncing his belief that the measured secondaries will normally be smaller than the calculated values.

The fact that this relation was fulfilled by the measurements taken in all the eleven erection stages preceding the closure of the arch, and that the exceptions are found only in Stages 3-*H* and 2-*H*, would seem to be a convincing corroboration of the writer's thesis.

If there were a satisfactory method of providing in the computations for the complicating effect of the transformation from cantilever to arch, and for any similar cause of augmented secondary stresses, then there would be no need of appending any qualification to the writer's conclusion as to the relative values of measured and calculated secondaries.

As a consequence of the above-described effect of the cantilever method of erection, the final stresses in the extreme fibers of the Hell Gate Arch chord members would have been seriously larger than the computed values, were it not for the provision of the three-faced joints; and this measure would have been of little value without the additional

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precaution which was adopted, of postponing the riveting until after the arch was closed.

If the joints had been riveted as the erection proceeded, instead of waiting for the closure of the span, it would have been possible to calculate the secondary stresses more definitely; the complicating effect of the transformation from cantilever to arch would have been obviated, and the final measured secondaries, in all probability, would not have exceeded the computed values. The only question is whether these advantages would have outweighed those which were secured by the use of the three-faced joints in conjunction with deferred riveting.

In connection with the points just discussed, Dr. Fuller asks for the results of the measurements of secondary stresses in Stage 3-*H*, and the writer is glad to supply these values in Table 9.

TABLE 9.—STAGE 3-*H*.
(All values are in pounds per square inch.)

Member.	Calculated S. S.	Measured S. S.	Member.	Calculated S. S.	Measured S. S.	Member.	Calculated S. S.	Measured S. S.
0-2	104	238	8-10	264	1 125	16-18	710	994
2-0	445	813	10-8	395	925	18-16	897	844
2-4	319	388	10-12	425	688	18-20	1 079	1 050
4-2	104	788	12-10	565	788	20-18	959	1 182
4-6	8	788	12-14	613	513	20-22	1 124	1 482
6-4	324	1 013	14-12	728	1 013	22-20	935	750
6-8	328	463	14-16	752	1 200	22-22 A	1 187	900
8-6	200	713	16-14	660	850	22A-22	258	563
Average							557	832

The averages shown at the foot of Table 9 are the values which were included in Table 5. It should be noted that the measured secondary stresses in Stage 3-*H* averaged 50% higher than the calculated values, as a consequence of the conditions discussed in the preceding paragraphs.

The erratic character of the individual results in this stage is to be attributed to the fact that the calculated and measured values do not refer to identical conditions of the structure. The calculated secondaries were obtained for the completed stage, with the entire floor erected, whereas the measured values were secured during early and varying stages of the floor erection. In consequence of the rapid progress made in this erection stage, as compared with the slow operation of taking the observations, the different measurements were obtained

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with changing positions of the erecting travelers and with varying quantities of floor system completed. Without the excessive labor of computing secondary stresses for so many different conditions of dead load on the structure, a proper comparison of calculated and measured values could not be submitted; for this reason, these percentages were omitted from the tabulation on Plate XLIV.

Dr. Fuller calls attention to the low percentages of computed secondary stress tabulated on Plate XLVII as contrasted with the higher values of the measured secondaries in the same stage (2-*H*) given on Plate XLIV. There really was no need of going to two different plates to find this relation, as it is frankly exhibited in direct comparisons in Plate XLIV. It is also indicated by the upper branch of the graph of measured secondaries on Fig. 5. It should be remembered, however, that these values of the measured stresses correspond to the usual theoretical definition of secondaries, as one-half the difference between the top and bottom fiber stresses; whereas, in this instance, on account of the effect of the three-faced joints, a more suitable definition of the secondaries is the excess of extreme fiber stress over the average stress in the section.

Ordinarily, these two definitions of secondary stress are equivalent; in the Hell Gate Arch chord members, however, the distinction is important, and it is evident that the second definition is the one of greater practical significance. In Fig. 5, the two methods of defining the measured secondaries are represented by the two branches of the graph for Stage 2-*H*; the divergence of these two branches shows the benefit secured by the use of three-faced joints, namely, to reduce the measured secondaries from an average of 220% down to an average of 58% of the calculated values.

The question is raised by Dr. Fuller as to the possibility of variation of stresses within the 20-in. gauge length that was used, and he suggests that smaller lengths are needed to catch the maximum values. This may be the case in short-span reinforced concrete beams and slabs, where, on account of the rapid drop in maximum stresses from the points of connection of members, it is found desirable even to reduce the customary 8-in. gauge length to 2 in. In the case of the 45-ft. members of the Hell Gate Arch, however, the variation of stress in a 20-in. length, at the points where the measurements were taken, was practically negligible, and the use of a shorter instrument would offer no advantage to compensate for the reduced accuracy of the results.

Investigators whose experience in strain measurements has been on concrete structures should remember that, on account of the difference in elastic coefficients, extensometer observations on steel require about fifteen times as great accuracy in order to secure the same precision in the values of the stresses; consequently, a longer gauge line is necessary in work on steel structures. Furthermore, in reinforced concrete spans,

the flexural stress declines rapidly, along a parabolic curve, from the end connection to the point of contraflexure; whereas, in steel chord members, the secondary stress has a linear variation which is very gradual in comparison. Mr. Steinman.

For the Hell Gate Arch investigations, both 10-in. and 20-in. extensometers had been provided; but the considerations just outlined prompted the writer to adopt the 20-in. instrument for the measurements on the main members, and to reserve the 10-in. instruments for observations on gusset-plates and on other connection details. It is regretted, however, that lack of time made it impossible to go very far with the latter measurements, or to take any observations on the distribution of stress between plates and angles composing a section.

The suggestion is made by Dr. Fuller that a Berry instrument might have given more satisfactory results than the Howard strain gauge, but he confesses that he has never used the latter. In the Department of Bridges, of New York City, the Howard instrument was adopted for all stress investigations, after consideration and trial of other types. With the Howard strain gauge, accurate results are secured by repeating the readings until they prove consistent and then recording the average. Although the Berry extensometer may permit quicker readings, it is doubtful whether the same accuracy is afforded, as a larger number of moving parts are involved. The Berry gauge gives all desired accuracy for getting stresses in concrete, on account of the lower value of the modulus of elasticity; and it has its advantage of speed for observations under changing load; but, for the work which was done on the Hell Gate Arch, it is believed that the Howard gauge was the more suitable.

Mr. Howard mentions some of the interesting possibilities of measurements with his extensometer, and he certainly merits the thanks of the Profession for his invention of the instrument and for developing its application. It is to be regretted that he has never found time to prepare, for the Society, an account of his measurements along the different lines to which they have been applied.

The writer communicated with Mr. Howard in reference to several points raised in these discussions and received the following letter in reply:

"INTERSTATE COMMERCE COMMISSION
"DIVISION OF SAFETY
"WASHINGTON

"DR. D. B. STEINMAN, "MAY 11TH, 1918.
"35 Nassau Street,
"New York.

"DEAR DOCTOR: Please excuse this tardy response to your letter of the 7th inst. I have been out of town part of the time.

"In regard to Mr. Powell's misgivings that the use of the strain gauge does not permit the necessary accuracy, I might mention that

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so far as check readings go they very often come out better than might be expected. Some readings taken on the lower chord of the Missouri River bridge at Kansas City in the forenoon and repeated in the afternoon of the same day chanced to accord to the same ten-thousandth throughout the series, of some twenty or more readings, corrections for temperature made in each case. The drilled and reamed holes in the bridge member are however rarely so well made that readings can be repeated as above. Usually the readings on the reference bar check, without variation greater than one-thousandth. The degree of accuracy depends upon the care exercised in drilling and reaming the holes. The holes should be smoothed with a conical set after drilling and reaming. A series of readings where successive changes in the loads occur very generally yield results which indicate the readings are reliable to one or two ten-thousandths. I do not make use of readings down to a ten-thousandth but place reliance upon the stresses to less than 500 lbs. per sq. in. If there was any need of greater refinement in the work I think it could be had by exercising more care. Differences in temperature limit the degree of accuracy in many cases. My impression is that the gauge readings are better than I claim for them. If Mr. Powell's experience is with plate gauges or with plug or ring gauges and he finds it difficult to attain an accuracy of a ten-thousandth of an inch then his aptness for fine measurements is not to be commended. No good machinist would think he had a satisfactory fit if it was not within a ten-thousandth of an inch on fine work. Sensitiveness of touch, in some places, is such that measuring to a ten-thousandth can be considered as more than ordinary work.

"In respect to holding the gauge in position normal to the work. Provided the drilled and reamed holes are strictly normal to the plane of the work, I do not think a slight inclination of the gauge would be found to give a change in the reading. The conical points of the gauge would make contact very nearly as a cylinder and center themselves on an elliptical contact zone. Reference lengths in the field are not paralleled as a rule and normal to the surface of the work. In order to avoid error in placing the gauge, it is my practice to use the same end of the gauge over the same hole in each observation, and judge of the gauge being normal to the work by noting the annular space around the conical point at the same end of the instrument on each occasion. One person should hold the instrument in place against the work, another advance the micrometer and make contact, and read the gauge. With suitable conditions, the holes being in good order, there is hardly any trouble in noting contact by sensitiveness of touch, with an accuracy of one ten-thousandth. Mr. Powell's suggestion of using a projecting pin with spherical end is not a practical one. Without going into the reasons why, if Mr. Powell will try such a method he will at once see the many difficulties which will defeat the accuracy easily attained with conical points and drilled and reamed holes. I doubt whether he would offer such a suggestion if he ever had experience with his suggested method. Of course the holes have to be cleaned at each reading. Dust has measurable thickness or diameter and must not be included with the steel member.

"In regard to Dr. Fuller's remarks about the use of a Berry gauge, which has a bell crank lever in its construction, strictly considered this is not a satisfactory detail of design. There is a difference between tilting a gauge of my type in which both points remain in the same plane and the action of a bell crank contact at one end. The rectilinear movement of the inner member in my gauge is mathematically correct. With this rectilinear movement there is no reasonable limit to the range of the instrument. I use a micrometer screw of one-half inch travel. It could be used with an inch travel or more if desired. The action of the gauge would not be impaired if the gauge length on the work was an inch longer or shorter than the reference bar. You will readily see how limited the use of a bell crank lever is in such an instrument as a strain gauge. The readings might be repeated and check in each case and still the measured length be in error. Parallel and rectilinear movements are desirable in instruments of precision. Mr. Steinman.

"I note your kind allusion to the desirability of hearing from me in some of the strain gauge work not yet published, and much regret that I have not done so. Perhaps at an early date I may have opportunity to prepare some notes on the Missouri River Bridge and some structural work done in New York.

"Yours very truly,

"(Signed) JAMES E. HOWARD."

Dr. Lindenthal explains the considerations which prompted him to undertake the measurement of the stresses in the Hell Gate Arch. In a review of the erection of the Eads' Bridge at St. Louis he presents a striking illustration of the possible serious effect of stresses created in a bridge during construction, and he thus emphasizes the importance of ascertaining the actual stresses left in a bridge after erection. He succinctly expresses the guiding consideration in his concluding paragraph:

"The object of the strain measurements, as related by the author, was fully attained. There are no unknown stresses in the Hell Gate Arch structure."

Dr. Lindenthal certainly deserves all possible credit for his initiative and public spirit in personally defraying the expenses of this investigation as a contribution to engineering science.

Mr. Delson presents an account of a very interesting series of extensometer observations conducted by the New York Department of Bridges in connection with one of the operations of strengthening the side spans of the Williamsburg Bridge. This experience of the Bridge Department afforded a striking instance of the value of the extensometer in such work, as the measurements supplied the information necessary for the solution of a perplexing and critical erection problem. The stress readings presented by Mr. Delson in connection with this

Mr. Steinman. incident also indicate the high precision attainable with the Howard extensometer.

Professor Parcel, in a very able discussion, raises some doubts as to the validity of two conclusions which he credits to the writer. The first of these conclusions is stated as follows:

"1.—Actual secondary stresses, in general, will be less than the computed stresses, due largely to a constant automatic re-adjustment of the internal strains tending toward the relief of secondary stresses."

This is a correct and clearly expressed paraphrase of statements made in the paper, but the writer wishes to make it clear that he did not offer this principle as a direct conclusion from the results of the investigation under discussion. It was merely an expression of a relation which he derived from outside considerations, and in the validity of which he had, and still has, firm conviction. It was interjected as a partial explanation of certain findings, and not as a direct deduction from the data of this experiment.

Professor Parcel admits that the principle is perhaps entirely sound, but he questions its consistency with the results of the measurements. On this point the writer has already presented his case in replying to the discussion of Dr. Fuller. For the sake of clearness, even at the risk of reiteration, it may be stated here that all apparent violations of the above-formulated relation in the comparison of the final results were associated with the unique and exceptional features in the Hell Gate Arch erection—features that do not occur in the usual cases of truss building. The most significant of these special features was the cantilever erection, resulting in the fixation of large secondary stresses at the joints during the cantilever stages, which stresses were not released in the transformation to the arch condition.

The special erection feature just described would naturally affect the results in Stages 3-*H* and 2-*H*, tending to increase the secondary stresses in those stages above the calculated values. This is exactly the phenomenon observed, for it is only in those stages that the measured secondaries exceed the calculated; and this fact, to which both Professors Fuller and Parcel call attention, instead of contradicting the writer's conclusions, serves as a clear confirmation of the correctness of his reasoning.

Apparently ignoring the above considerations, Professor Parcel thinks that the following additional conclusion might fairly be drawn: "For the higher ranges of primary stress, the actual secondary stresses will probably exceed the calculated values by a considerable margin." To this conclusion the writer takes distinct exception. It is based on a comparison of the results in the arch stages with those in the cantilever stages—a comparison which yields misleading conclusions, or

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conclusions the generality of which is vitiated, on account of the special considerations already submitted. The only comparisons from which a conclusion of this character could legitimately be drawn would be between measurements of stresses of different magnitudes in the same stage or condition of the structure; and such comparisons, in the present investigation, distinctly contradict Professor Parcel's suggested conclusion, and confirm that of the writer instead.

The writer maintains that the apparent exception in Stages 3-*H* and 2-*H* to his rule, that actual secondary stresses will generally be less than computed stresses, is due merely to the fact that in these stages there was a gross but unavoidable incorrectness in the calculated secondaries. He regrets the inadequacy of the available methods of computing secondary stresses to take into account the effect of such complicating factors as the transformation of the structure from a cantilever into an arch. If a satisfactory solution of this mathematical problem were obtainable, there is little doubt that the anomalous results under discussion would be entirely dispelled.

It may be suggested here that the problem just described would be simple of solution if the joints were securely riveted before the transformation. It is the partial and unknown slipping or yielding of the joints before or during riveting that complicates the theoretical problem.

The second conclusion presented by Professor Parcel for discussion is:

"2.—For most bridges it will probably be satisfactory to provide for secondary stresses by an allowance in the specified unit stresses of about 20% of the total primary stress, plus 3 000 lb."

A comparison of this version with that of the writer in Conclusion 4, on page 1071, will indicate certain differences. The writer did not suggest the deduction of 25% plus 4 000 lb. from the elastic limit as a "satisfactory" allowance but as a minimum "necessary" allowance. In doing this he was merely calling attention to the necessity, in fixing extreme values of working stresses, of leaving a sufficient margin below the elastic limit to provide for secondaries and other anticipated additions to calculated primary stresses; and he wished to suggest the possible usefulness of investigations of this character in supplying information to help fix the proper magnitude of this margin or allowance. The value suggested by the writer was simply the best one that he could deduce with the aid of the data of this experiment; and it was offered for whatever it was worth, merely as an initial contribution to the fund of knowledge requisite to form a proper judgment on this point. No one realizes better than the writer the importance of securing similar data from measurements on a great number and variety of structures before any final decision on such a question can be reached.

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The writer's reasons for including this matter in the paper were twofold: First, to indicate an additional objective for extensometer and secondary stress investigations; and, second, to suggest a form for an empirical rule for a margin above extreme working stresses.

In the original presentation of this tentative rule, on page 1063, it will be observed that judgment based on outside considerations generously supplemented logical deduction in the formulation of the conclusion. It was more in the nature of a suggested line of thought and future investigation than a final and direct conclusion from the data of the investigation.

That the limitations and present indefiniteness of such an empirical rule were clearly realized by the writer is shown by the following quotation from the paper:

"It is recognized, of course, that a rule of this form can properly apply only to structures in which the secondary stresses have normal or average values. Where unusually large secondaries may be anticipated, these should receive special investigation and attention."

Professor Parcel raises the question "whether any empirical relation based on averages is an adequate means of providing for secondary stresses, even in ordinary structures." The writer endeavored to make his proposed allowance sufficiently generous to take care of the maximum, and not the average, secondary stresses ordinarily occurring. That the suggested allowance is adequate is corroborated by the results of Professor Parcel's own measurements of secondary stresses as plotted in Figs. 9, 10, and 11. These graphs show that the secondary stresses in a railway bridge under service loading ranged between 10 and 30%, and never exceeded the latter amount except when the primary stresses fell very low; and, as Professor Parcel himself remarks, "it is only the secondary stresses that co-exist with the maximum primary stresses that affect the design of the structure."

The writer does not take issue with Professor Parcel's suggestion of the desirability of making some analysis of the secondary stresses in every design of a bridge structure. That, indeed, would be a most desirable consummation; but it is going to be very difficult to convert the Profession to such procedure in the case of ordinary structures, or to get clients to pay for the additional labor involved.

In proposing the use of empirical allowances for secondary stresses, the writer was not actuated by any distrust of the reliability of secondary stress computations, nor by any doubt as to their desirability, but rather by a conviction of the difficulty of changing present practice in this regard. So long as engineers have to deal with actual conditions where secondary stress computations are generally omitted, it is certainly better to have some guide for estimating a proper allowance than to get along without any. Therefore the writer offered, as a

suggestion, a rule for this purpose, derived from his best judgment, guided by the results of this investigation. No data tending to contradict or modify this rule have yet been advanced. Mr. Steinman.

Professor Parcel writes:

"If we assume that theoretical computations for secondaries are wholly untrustworthy, and thus justify reliance on an empirical formula for ordinary bridges, how are we to rely on our analysis for the cases where 'exceptionally large secondaries may be anticipated', and what basis is there for the elaborate and very expensive correction methods used in the erection of the Quebec and Sciotoville spans, methods which have been referred to in current technical literature as the most remarkable single feature of these great engineering projects?"

In reply the writer wishes to make it clear that he does not regard theoretical computations for secondaries as untrustworthy; on the contrary, he is convinced that, with the exception of cases involving complicated erection features, which cannot be accurately considered in the computations, the actual secondaries will be either equal to or slightly less than the calculated values. The use of an empirical formula is recommended, not as being more reliable than theoretical computations of secondary stresses, but as constituting a more or less satisfactory substitute when time and labor have to be conserved.

Referring to cases where "exceptionally large secondaries may be anticipated", the writer submits that such cases can readily be recognized by any one who has had proper experience in the study of secondary stresses. Exceptional stresses will be found at points where there is any interference with the production of a smooth curve of deformation of a span or of a single member. Such points are: at the feet of main hangers in simple Pratt trusses; at the feet of sub-hangers in trusses with subdivided panels; at the connections of collision struts; at the points of attachment of back-stays in cantilever erection of arches; at intermediate supports of continuous and cantilever bridges; at the connections of intersecting diagonals, etc. Fortunately, in nearly all such cases, it is possible to ascertain the approximate value of the critical secondaries by some simplified method based on local deflections without undertaking a complete theoretical analysis of the secondary stresses in the entire structure. It would seem that such procedure, in combination with the use of an empirical rule for the secondaries of ordinary magnitude in the structure, would provide a safe and economical solution of the problem of providing for secondary stresses.

Professor Parcel refers to the elaborate and expensive erection methods which were adopted for the elimination of the secondary stresses in recently completed long-span bridges. In the case of the Sciotoville Bridge, the writer can speak from experience, as he was

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identified with the design and computations for that structure. The Sciotoville Bridge presents a striking instance of a case where "exceptionally large secondaries may be anticipated." Over the middle support of this continuous structure, there is necessarily a large upward cusp in the curve of deflections, and the secondary stresses from this cause, for live load on both spans, would have amounted to nearly 20 000 lb. per sq. in. Although elaborate secondary stress computations were prepared for the entire structure, the critical stress just mentioned was easily foreseen; and it was computed in a few minutes from the deflection diagram of the trusses, quite independently of the more complete analysis of secondaries. Similarly, the secondary stresses in the bottom chords at the connections of the various subhangers could have been foreseen and computed independently by approximate methods. Such approximate investigations would have been sufficient to disclose the serious character of the secondary stresses in this structure, and to emphasize the necessity of some such method of erection as was adopted to counteract these dangerous stresses.

Professor Parcel adds:

"We need a great many more experimental data to settle the question of actual *versus* calculated secondary stresses, no doubt; but the bulk of the evidence, up to the present, would seem to justify considerable confidence in our theory."

It is difficult to understand what is referred to in the phrase "the bulk of the evidence, up to the present"; since, as the writer endeavored to point out in the introductory part of the paper, there had been practically nothing published in the form of a comparison of calculated and measured secondary stresses before the presentation of this investigation. Nevertheless, the writer joins Professor Parcel in placing considerable confidence in the theory of secondary stresses; in fact, it does not appear that any one has questioned it.

The results of stress measurements on the Kenova Bridge, presented by Professor Parcel, are certainly interesting. It is unfortunate that the data have not yet been worked up to show a comparison between measured and computed secondaries, and it is to be hoped that this analysis will soon be completed and presented to the Profession.

The writer is grateful to Professor MacKay for directing attention to a point of possible misunderstanding, namely, the statement in the paper about the number of measurements required to determine the distribution of stress in any member. When he wrote "It is necessary to take measurements at not less than three, and preferably four, points of the cross-section near each end of the member", the writer intended this merely as a statement of minimum requirement, to apply only to members of ordinary size and section; and he is in complete accord with Professor MacKay in considering that a greater

number of points are usually desirable in large members. The writer recognized this fact in the case of the Hell Gate chord members, of which he wrote (page 1052), that "measurements at the four extreme corners of the section would not be fairly representative of the conditions throughout the entire area." Mr. Steinman.

The figures cited by Professor MacKay from the Hell Gate measurements to show that the omission of the readings near the center of the webs would have altered the results, serve as a confirmation of the writer's judgment in deciding to use six observations at each cross-section. The two readings at mid-height were necessary, not only on account of the mass of metal concentrated at and near the middle diaphragm, but also to determine and provide for the effect of the concentration of bearing in the middle thirds of the joints.

Whether a larger number than six observations at each section should have been taken, it is difficult to decide. It is doubtful whether there would have been any appreciable increase in the reliability of the results to compensate for the curtailment of programme which the extra labor would have necessitated.

The data of tests on latticed members submitted by Professor MacKay are extremely interesting. His plats of stress distribution near the end of a pin-connected member show a concentration of stress toward the center of the web, as might be anticipated. He also shows the disturbance of uniform stress condition at the points of connection of diaphragms and lattice bars, on account of local bending in the ribs composing the member. It was in anticipation of such effects that the writer avoided the proximity of diaphragms, splice-plates, and similar factors of stress disturbance, in fixing the locations of the gauge points for his stress measurements on the Hell Gate Arch.

In correspondence with the writer, Professor MacKay described some of his measurements on bridges of the Canadian Government Railways, and it is to be regretted that these came too late to be included in his published discussion. With Professor MacKay's permission, the following is quoted from one of his letters:

"My work on the C. G. R. bridges was limited to a single type of riveted pony truss span, and my object was to check primary, secondary, and impact stresses under live load. The results would dispose me to endorse all of your conclusions applicable to such widely divergent cases, with the possible exception of the point referred to in my discussion."

The following are some of the results as regards precision in the span most thoroughly examined. The range of stress measured was from 5 000 to 8 000 lb. per sq. in.

Difference between calculated and observed primary stresses:

Average.................390 lb. per sq. in. (525)

Maximum................780 " " " " (1 290)

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This omits some lower chord members which were affected by stringer and floor-beam action, and by the failure of roller bearings to work.

Difference between stresses in two ends of same member:

Average ....270 lb. per sq. in. (140)
Average, omitting one section.......160 " " "
Maximum ....960 " " " (500)
Maximum, omitting one section.....320 " " "

Most of the work was done under favorable temperature conditions.

As regards secondary stresses, the most important effects agreed fairly well with calculated stresses.

The figures in parentheses, representing corresponding values from the Hell Gate Arch investigation, have been inserted by the writer to permit a comparison with the results secured by Professor MacKay. When the difference in the characters of the structures and loadings is considered, the two sets of results may be regarded as mutually confirmatory.

In the discussion by Professor Turneaure, we have a clear presentation of the significance of secondary stresses by one who is an acknowledged authority on the subject. He writes that he "does not believe that it will ever be necessary or desirable to calculate secondary stresses in the ordinary practice of bridge design; but he does believe that their consideration, both theoretically and experimentally, in such unusual structures as the one under consideration, is of very great importance."

It was in connection with the ordinary cases of bridge design, for which no calculation of secondary stresses is undertaken, that the writer suggested the use of an empirical correction to be applied to the working stress. Referring to the suggested figure of 25% + 4 000, Professor Turneaure agrees that this "would probably cover most cases of well-designed structures." In arriving at this estimate of a proper deduction from the minimum elastic limit, the writer took into consideration, not only the secondary stresses usually to be expected, but also other possible variations from computed stress.

The writer, as previously stated, did not intend to present the foregoing recommendation as a direct and rigorous conclusion from the results of the calculations and measurements on the Hell Gate Arch. Other considerations entered in arriving at this figure, and it was offered merely as a guide or basis for future investigation and discussion. Consequently, the following statement by Professor Turneaure on the possible value of the suggestion is appreciated:

"If it is found that the secondary stresses in any particular structure can be represented safely by a particular percentage of the primary stresses or a percentage plus a constant, then we have arrived at a

fair value of the maximum fiber stress, and are much better prepared to discuss the subject of working stresses than if no estimate of the secondary stresses had been made." Mr. Steinman

In reference to the reliability of secondary stress calculations, Professor Turneaure writes:

"Such stresses are just as much a real part of the total stress which may prevail in any fiber of a member as the primary stress; and they are just as certain to exist as it is certain that the structure will deflect under load. The accuracy of their determination, however, is not so great as that of the primary stresses, on account of the indeterminate effect of the temperature variations and of joint plates, rivets, and other details."

It is on account of this uncertainty which characterizes secondary stress calculations that the writer recommends the application of the extensometer to the actual determination of the fiber stresses in all structures of unusual magnitude or construction.

Referring to the great care taken in the design and erection of the Hell Gate Arch and of other large structures as evidence of the increasing appreciation of scientific methods of design and accurate fabrication, Professor Turneaure writes:

"Where such conditions prevail, the objection to the use of statically indeterminate structures, which has been so general in the past, loses its force, and there would seem to be no reason for excluding such forms of structure where, for other reasons, they are desirable."

This statement cannot be too heartily endorsed; it expresses what is probably the most important lesson to be learned by the Profession from the work on such structures as the Hell Gate Arch and the Sciotoville Bridge.

Professor Jacoby emphasizes the value of scientific investigation as conducive to security and economy in structural design. He adds:

"When observations are made on an actual structure, especially one in which the members are so large and where the resources of construction in equipment and workmanship are taxed to a much higher degree than usual, the results are of far greater value than laboratory experiments could possibly give."

The realization of this opportunity in connection with the erection of the Hell Gate Arch was the incentive for undertaking the stress measurements, and the justification of the financial outlay on the part of Dr. Lindenthal.

The summary of the investigation of secondary stresses in a two-hinged, spandrel-braced arch with cantilever arms, computed by Mr. Thomson Mao, of the Cornell University Graduate School, and presented by Professor Jacoby, constitutes a valuable contribution to this discussion. It exemplifies the diversity of methods applicable

Mr. Steinman.

to the calculation of secondary stresses. In Mr. Mao's thesis, Mohr's method was used, involving the use of displacement diagrams to find the change in slope of the truss members. The method of Winkler, in which angle changes are found analytically, is preferred by the writer, because of its greater precision and ease of checking.

Another distinctive feature in the method of computation followed by Mr. Mao was the calculation of the secondary stresses in all the members of the structure for a unit load at each of the panel points. Except for spans having a very small number of panels this procedure would involve too great an expenditure of time and effort for application in actual practice. All essential purposes of secondary stress computations will be served if the analysis is made for a much smaller number of cases of loading. As a rule, the simplest considerations of the natural deflection characteristics of a structure will indicate the proper loading assumptions for maximum secondary stresses; and an inspection of the load lengths for maximum primary stresses will indicate the one or two loading assumptions which will give, with sufficient accuracy, the critical secondary stresses, namely, those occurring simultaneously with the maximum primary stresses.

Relative to this matter, the writer would offer, as a recommendation for future secondary stress computations, the suggestion to compute the stresses for just two load conditions, *viz.*, the first and the second quarters of the span, respectively, loaded. By simple reversal of the diagram and algebraic addition of the sets of stresses, there will be obtained the secondary stresses for load covering one-quarter, one-half, three-quarters, or full span, or any other combination of quarters of the span. The resulting values will suffice for all practical purposes, and by this procedure the analysis will involve only two solutions of secondary stresses, instead of the ten or more required in Mr. Mao's work.

Professor Jacoby recommends the method of Gauss for the solution of the sets of simultaneous equations involved in the computations. He writes:

"By the method of Gauss, the solution of the equations becomes quite mechanical, and may be checked at every step of the process. This method of solution, therefore, has a great advantage over any other."

The writer doubts whether this method can compare favorably in speed with the procedure of successive substitution, as described on page 1073, which was used for the Hell Gate computations, and yielded a remarkable showing over all other methods in its greater speed and self-checking advantages.

On this point, the writer has had some correspondence with Professor Turneaure, who advocated an improved method of successive elimination as being the most expeditious. To this the writer replied:

"I believe that if you include the time required to check the work, to locate errors and to rectify them, the method of elimination will be seriously handicapped in comparison with the methods of successive approximation by substitution. The figures which I gave you in my last letter for the time required by the different methods included the work of retracing the steps whenever an error was suspected, and of correcting any such error. If we consider the primary desideratum to be correctness of the results, rather than merely securing of maximum speed, it seems to me that the method of substitution is the more desirable, as it supplies an automatic check at every step in the work. It is a self-checking method in that it is impossible to proceed very far without detecting any error that may have been made." Mr. Steinman.

The writer has worked out a systematic arrangement of the computations in the method of successive substitutions so as to minimize the consumption of time and the possibility of mistakes. In actual test he has not yet found any other method to equal it in the desiderata of speed and self-checking.

The writer is grateful to Mr. Ammann for calling attention to the possible danger of applying the conclusions of this paper without due regard to special conditions which might modify the results.

In discussing the findings of his investigation, the writer has been careful to keep in mind at all times the special features in the design and erection of the Hell Gate Arch which would affect the results in any way. He submits that a careful perusal of the ten conclusions presented at the end of the paper (pages 1070 to 1072), will show no violation of this principle.

There is but one paragraph occurring in those pages, which is not a rigorous deduction from the results of the investigation; but its very form indicates that it was not presented as a deduction or conclusion, but rather as a parenthetical adduction of a conviction entertained by the writer and tending to throw light on certain results obtained. It refers to the observed fact that most of the measured secondary stresses during the erection of the arch were less than the calculated values, and reads as follows:

"It is believed that similar results, though not as marked, would be found in other structures. The actual secondary stresses will generally be lower than the calculated values. There is an automatic re-adjustment of strains within a structure in such direction as to relieve the secondary stresses."

The writer trusts that no one will take exception to the foregoing statement, as it is a simple matter to demonstrate, both mathematically and practically, the validity of the proposition therein enunciated. It is not a conclusion from the present investigation, although it is corroborated by the results obtained.

Mr. Steinman.

Apart from this passage, which was not intended as a conclusion, and from Conclusion No. 4, which has already been the subject of extensive discussion, the writer would like to ask Mr. Ammann to point out anything in the summary of conclusions which can possibly be regarded as too far-reaching a deduction from the results of the investigation.

Mr. Ammann refers to the objects for which the measurements were undertaken, but this matter has already been presented in the paper and in Dr. Lindenthal's discussion.

Mr. Ammann points out that one of the objects in the programme of investigation has not been accomplished, namely the measurement of the live-load stresses in the finished structure. It is to be regretted that Dr. Lindenthal interrupted the investigation at this critical point, and the writer joins Mr. Ammann and the others who have expressed the hope that some Federal bureau or technical society may take up the measurements, so as to include the final live-load stresses.

Mr. Randorf contributes an important suggestion as to the possible influence of the varying elasticity of the steel on the results of the measurements. Although a uniform value of 30 000 000 for the modulus of elasticity was assumed in reducing the observations, the effect of possible variation from this value was discussed on page 1061 of the paper.

Professor MacKay, in his investigation, as mentioned on page 1095, adopted a value of 29 000 000 for the elastic modulus. The difference in the quality of the steel in the two structures tested would justify this difference in the assumed values of E.

Mr. Randorf's suggestion that the modulus of elasticity of the material should be verified by special tests is certainly a good one. In the present case, the satisfactory agreement between average calculated and average measured stresses would appear to be a confirmation of the correctness of the assumed value of E; nevertheless, it is readily conceivable that some of the variations in the results might be traced to fluctuations in the elasticity of the material.

Most of the questions raised in Mr. Molitor's valued contribution have already been answered in connection with the preceding discussions. There are a number of points, however, which call for individual consideration.

Referring to the features which gave rise to the observed discrepancies between calculated and measured secondary stresses, Mr. Molitor wants to know why these features were not provided for in the computations, so that a closer comparison might have been secured. Perhaps the best way of answering this question is to present for

individual consideration the following list of factors tending to produce deviations from computed secondary stresses: Mr. Steinman.

1.—The common theory of secondary stresses assumes absolute angular rigidity of the connections, whereas, actually, there must be some elastic yielding or mechanical slip between the members. This condition, occurring to some extent in all structures, tends to relieve the secondary stresses. The amount, however, is unknown, and is not susceptible of analytical determination. The effect is augmented materially by the slip and re-adjustment which take place when the drift-pins and bolts used temporarily during erection are replaced by rivets. No attempt is made to correct for this condition in the computations, as the part due to elastic strain is too small to justify the effort, and that due to mechanical slip is too erratic and arbitrary to permit analysis. It is, of course, on the safe side to ignore this effect.

2.—The common theory of secondary stresses neglects the bending moments produced in the members by the eccentricities resulting from their flexural deformations. This condition tends to augment the secondary stresses in compression members and to reduce them in tension members. Provision for this effect greatly complicates the analysis and computations, and ordinarily it may be disregarded. For long and slender members, however, the effect would be appreciable.

3.—In the generally accepted method of analyzing trusses, the primary stresses are determined on the basis of frictionless joints. There is a small error involved in this assumption, as the joints are not frictionless, but possess rigidity. The elastic curvature assumed by the members under the secondary strains gives them a girder action whereby a certain amount of transverse shear is transmitted from panel point to panel point, and the primary stresses are relieved to that extent. This, in turn, results in a slight modification of the secondary stresses, generally a reduction; but the effect is small, and as the corresponding calculations are laborious, they are neglected.

4.—Shop inaccuracies in laying out the rivet holes for end connections, either as to distance or angle, will produce secondary stresses not covered by the computations.

5.—Temperature differences between parts of the structure or of the same members will produce secondary stresses not included in the calculations.

6.—Inaccurate facing of the ends of compression members may produce eccentric bearing at the butt-joints, and the effect will appear in the measured secondary stresses.

7.—An important factor is the use of drift-pins during erection. When these are replaced by rivets, a certain amount of mechanical slip and re-adjustment of strain occurs at the joints, permitting a

Mr. Steinman.

partial release of the secondary strains. If the secondary stresses then obtaining are contrary in sign to the final secondaries at the same points, the ultimate effect of the mechanical slip will be an augmentation of secondary stress. On the whole, this feature is too erratic and arbitrary to permit analysis; consequently, it is not provided for in the computations.

8.—For large, built-up sections, there is some uncertainty as to the distribution of stress over the cross-section. Navier's law cannot be assumed to apply with positiveness. The division of stress between the plates and angles composing the section may be imperfect, depending on the riveting; and local stress concentrations are produced by splices, diaphragms, and details. All these conditions affect the measured secondary stresses, but cannot be provided for in the computed secondaries.

The foregoing list of disturbing factors should serve to make clear why the exact values of the secondary stresses cannot be anticipated by computation, and why the results of theory should be supplemented by experimental investigations aiming to throw light on the individual and collective amounts of these deviations from the calculated stresses.

In addition to the conditions previously outlined, which are of more or less general occurrence, there were, in the case of the Hell Gate Arch, two other features affecting the secondary stresses, as follows:

1.—The three-faced joints: With regard to these, it is important for clearness to distinguish between their action in relieving the primary stresses in the outer fibers, and their influence in permitting partial hinge action at the connections during erection. The former effect does not change the secondary stresses (defined as one-half the difference between top and bottom fiber stresses); consequently, no correction for it is necessary before comparing measured and calculated secondaries. The hinge effect, on the other hand, does modify the secondary stresses, but it is more or less indeterminate qualitatively as well as quantitatively. It is resisted to an unknown extent by the presence of the splice-plates and drift-pins; and it ceases to occur when the primary stresses attain an undefined value. The ultimate effect on the secondaries will depend on the reversals of secondary flexure taking place during erection. For these reasons it would be practically futile to endeavor to provide for this hinge action in the computations.

2.—The cantilever erection: This produced large secondary stresses during the erection stage, and a part of these persisted in the structure after its transformation into an arch.

The effects of the ten features just enumerated as influencing the secondary stresses are complicated by their action in combination. The action of the three-faced joints is modified by the interference

of splice-plates and drift-pins; the effect of the cantilever erection is controlled by the unknown amount of hinge action taking place as a result of the three-faced joints; the replacement of drift-pins by rivets is accompanied by an uncertain amount of slip and strain redistribution in the connections; and the transformation from cantilever to arch involves unknown re-adjustments of strain at all the joints. Mr. Steinman.

In his computation of the secondary stresses, the writer followed the established procedure and did not provide for the previously discussed modifying factors. To take all these features into account analytically would be impossible; for most of them even a qualitative prediction would be difficult. The best that can be done is to determine their combined effect experimentally for individual cases, until sufficient data are thus accumulated to form a basis for empirical rules to be applied to other structures.

Mr. Molitor suggests that the features just discussed would necessarily result in very small secondary stresses. If he had predicted, in advance of this investigation, that the combined effect of these features would be to reduce the secondary stresses, he would have proved himself a poor prophet; for, as it happens, the secondary stresses in the final stage average about 120% greater than the corresponding computed values. (See Table 5.) He would have been underestimating the intensifying effect of the cantilever method of erection, which, in this case, outweighed the combined influence of all the other features tending to modify the secondary stresses.

What Mr. Molitor apparently has in mind is the easily anticipated relief in the stresses in the outer fibers as a result of the greater concentration of pressure in the middle thirds of the three-faced joints. This, however, is not a secondary stress in the accepted sense of the term; it is merely a redistribution of the primary or axial stress over the cross-section. In the case of the Hell Gate chord members, this action of the three-faced joints proved to be a saving feature; it just about neutralized the resultant augmentative effect of all the other disturbing conditions, so that the final extreme fiber stresses came down to the calculated values (see page 1068). For the sake of clearness, however, it should be understood that this effect of the three-faced joints, although representing a reduction of the outer fiber stresses, is not, strictly speaking, a reduction of the secondary (or flexural) stresses. It does not affect the comparison between measured and calculated stresses so long as these are defined as one-half the difference between top and bottom fiber stresses.

No exception can be taken to the point made by Mr. Molitor that the Hell Gate Arch Bridge is not the most suitable structure for proving or disproving the correctness of secondary stress computations; but that was not the object of these investigations. The

Mr. Steinman. measurements were undertaken primarily for the purpose of ascertaining the actual stresses in the Hell Gate Arch, and that object was attained. Incidentally, the results secured have been studied and the conclusions reported in order to throw whatever light they can on the effects of the various features tending to modify the stresses.

In this connection, the writer wishes to point out that, although the Hell Gate Arch results represent the composite effect of many stress-modifying elements, nevertheless, the measurements may be studied so as to single out the effects of some of the special features. For example, by comparing the results in the arch stages (3-*H* and 2-*H*) with those in the preceding cantilever stages, information is secured on the effect of the transformation from cantilever to arch; and it is thus found that this single feature of the erection outweighs the combined effect of all the other factors modifying the secondary stresses. Similarly, a comparison of the final secondary stresses according to the respective definitions represented by the two branches of the graph in Fig. 5, yields directly the effect of the three-faced joints in relieving the outer fiber stresses. With these two special features (cantilever erection and three-faced joints) separated out, there is little left in the results and conclusions that may not be of general application to other structures.

Referring to the method used for computing the secondary stresses in the Hell Gate Arch, Mr. Molitor remarks that it is "admittedly laborious, and might have been somewhat simplified". If Mr. Molitor knows of any solution that is less laborious, or of any further simplification that might be introduced, the Profession would certainly welcome his contribution. If, however, he is referring to the method described in the chapter on Secondary Stresses in his book, he is claiming an advantage which is not substantiated by actual comparisons. That solution was invented and published by Müller-Breslau more than 30 years ago, but it has failed to stand the test of time in competition with other methods.

The writer uses Winkler's method which involves only one unknown for each panel point, whereas Müller-Breslau's treatment requires the solution of two unknowns for each member. Thus, for the Pratt truss worked out by Mr. Molitor in his book, he has to determine 22 unknowns; for the same problem, Winkler's method would involve only 6 unknowns. The simultaneous equations in Winkler's solution are one for each panel point, while Müller-Breslau's method requires in addition three equations for each truss triangle; and the method of elimination used for solving these equations is no more expeditious than the procedure of successive approximations used by the writer.

In the discussions there has been practically no mention of two of the most interesting facts established in the course of the investigation.

One was the hinge action of the three-faced joints, permitting, in the early stages, a certain amount of rotation which was easily measured. The other was the re-adjustment and re-distribution of stress at the joints when the drift-pins were replaced by rivets. Before the measurements were made, doubt was expressed in certain quarters as to the actuality of both of these anticipated effects, but the investigation has set all such doubts at rest. Mr. Steinman.

In conclusion, the writer wishes to repeat his acknowledgment of indebtedness to all those who have discussed the paper, as he realizes that their contributions have greatly enhanced its usefulness to the Profession. On account of the necessary brevity of the paper, some of the thoughts and conclusions were perforce inadequately presented; and the discussions so generously contributed have helped to focus attention on the points of greatest interest and to remove any doubts as to the interpretations to be drawn.

This paper would not be complete without a closing tribute to Dr. Lindenthal for his foresight in conceiving the investigation, and for his public spirit in undertaking it at his own expense.

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Paper No. 1419

ICE DIVERSION, HYDRAULIC MODELS, AND HYDRAULIC SIMILARITY*

BY BENJAMIN F. GROAT, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. M. O. LEIGHTON, J. WALDO SMITH, ROBERT FLETCHER, A. F. PARKER, E. E. R. TRATMAN, AND BENJAMIN F. GROAT.

SYNOPSIS.

This paper treats of a new method of diverting surface water and all floating materials carried thereby for the purpose of preventing jams in canals and rivers. The paper also shows how hydraulic works may be designed by studying the performance of small-scale models.

ICE DIVERSION.

On many of our northern rivers, notably the Niagara and St. Lawrence, there is an immense annual crop of ice which must be taken care of in one way or another, in order to protect the various interests along the shores against damages resulting from shoves, jams, and heavy runs.

These heavy floes consist of ice in all its forms and conditions, so that no one method of treatment is likely to be entirely effective against all of them. There is, however, one property, more or less pronounced, which is common to all forms, and this fact is available toward a general solution of the problem. Ice is buoyant unless weighted down by heavier solids, such as stones frequently carried by

* Presented at the meeting of January 2d, 1918.

the anchor form. Even when no heavier solids are carried, there are forms of such spongy character that the water contained by the interstices produces a water-logged variety, which, once submerged, rises to the surface very slowly.

Some years ago it occurred to the writer that the proper way to divert floating materials is to make the surface currents carry them away, rather than use a boom, which, in reality, opposes, instead of assists, the movement of floatage, and has little or no effect at all on materials suspended below the surface of the water. Following this line of thought, a patent has been applied for which covers both method and means for effecting a diversion of floating materials superficially, and a diversion of water sub-superficially, so that all, or nearly all, the water containing ice can be diverted in one direction, while the remainder can be turned into another—as into a power canal—practically free from ice and all other floating débris. In addition, it is possible to prevent jams and to control the surface currents of a river or stream so that it will be capable of carrying off all floating materials as fast as they are supplied from above, or can form within it.

As it is principally the surface currents which carry floating materials, it will be possible to measure, more or less exactly, the transporting capacity of a river at a particular place by means of the product of the width and mean surface velocity at the place. For brevity, this product may be called the "transportivity" of the river at the given place.

The transportivity of a river, then, furnishes a test of the probability of a jam or congestion of ice at any place. If it be found, for example, that the transportivity of a stream, relative to the number of square feet of floatage per second to be discharged, is great at one point and small at another a short distance below, it would appear likely that a jam might form at some intermediate place. Such a condition would exist in a reach of a river which consists of a wide deep pool fed by a broad shallow section and discharged by a narrow deep outlet of relatively large sectional area. Evidently, the pool might be supplied with floating materials more rapidly than it could discharge them, the transportivity of the feeder being much greater than that of the outlet, saying nothing of any ice which might originate in the pool itself.

The foregoing statement is not based merely on theory, but rests on firmly established facts connected with the winter conditions obtaining on our northern streams generally.

The method, then, by which a diversion of water from a river to a power canal can be made, while all the ice is carried away by the river or main stream, is to cause the river to have a high surface transporting capacity in the vicinity of the canal intake and at the same time reduce to small proportions, or even make negative, the transportivity of the canal intake itself. It is clear that no material increase in the probability of jams below the intake will result by reason of this arrangement.

Fig. 1 shows the writer's method and means for securing this important result. A model of an actual river has been constructed to a scale of one-hundredth the full size. This reach is marked *SS'*, and shows the water flowing from right to left. The ice, represented by cakes of paraffin cut to scale, flows with the surface water. The intake of the canal is shown at *C*. In this particular instance, the entrance, or intake, of the canal, is too wide, and the first step toward reducing the transportivity of the intake is to introduce a jetty, *B*, extending part way across the opening. The transportivity of the intake may now be further reduced by dredging, or otherwise excavating, several channels in a transverse, or oblique, direction across the main stream and leading toward, to, or even into, the intake of the canal. As the channels must be separated by ridges, or other elevations between them, the transportivity of the main stream will not be altered materially if the crests of the ridges are left at the original surface of the bed of the stream. This condition must maintain because the effective transverse cross-section of the main stream has not been altered materially though the effective transverse cross-section of the intake has been enlarged to any desired extent. If it should be required to increase the transportivity of the main stream, the ridges may be built to any desired elevation higher than the original bed. This joint control of the surface transporting capacities of stream and canal intake can be exercised by adjusting proportions *ad libitum*.

In many cases it will be found necessary to provide a wing, *W*, to prevent materials from floating into the intake around the up-stream head, and it may be of advantage to construct a jetty, *D*, extending from the shore opposite the intake, or to build some other kind of

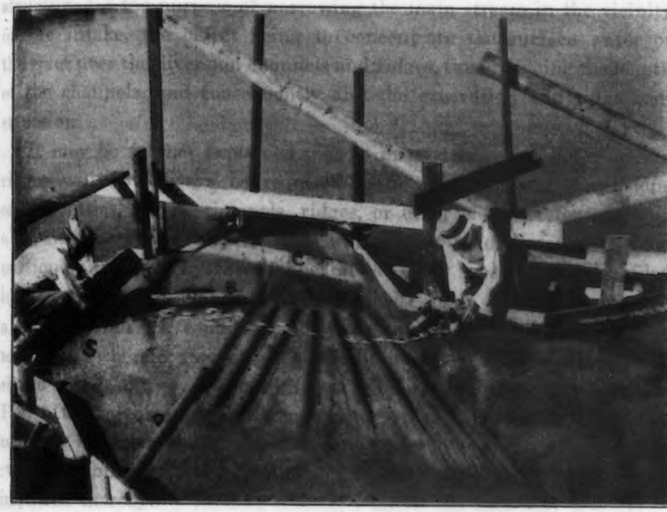

Fig. 1.—Model Illustrating Ice Diversion.

Fig. 2.

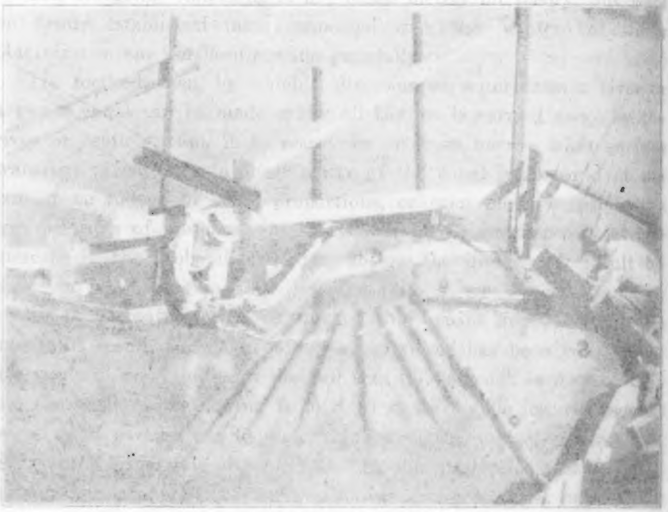

FIG. 1.—MODEL ILLUSTRATING ICE DIVERSION.

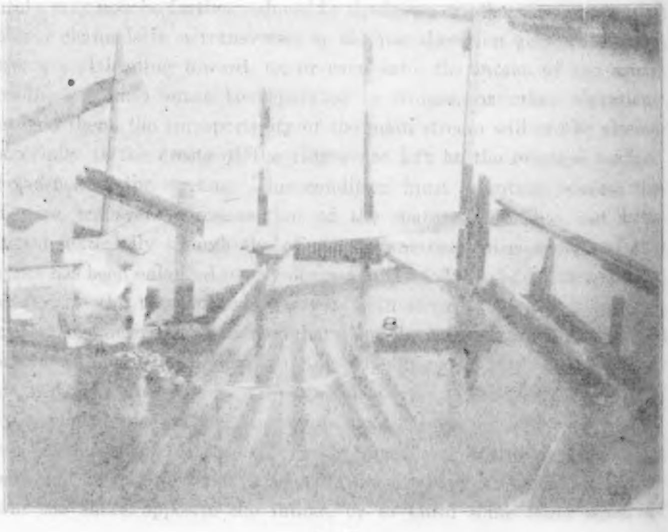

FIG. 2.

structure for the purpose of narrowing the main stream in the vicinity of the intake, the object being to concentrate the surface water of the river over the diversion channels and ridges, thus reducing the length of the channels, and consequently also the excavation and ridge construction.

It may be further explained that so many transverse channels and ridges will not always be required. The locations and dimensions of the jetties, wings, channels, ridges, or other elements for effecting a separation of the ice and canal water, must be determined in each particular instance by careful theoretical study most effectively aided by digesting the results of tests on models. As an illustration of such a change from the conditions shown in Fig. 1, the writer may refer to Fig. 2, wherein the jetty, *D*, has been removed. The illustration shows the result of a test with the same hydraulic conditions as in Fig. 1, the ice diversion being nearly as satisfactory, but the wing, *W*, requires considerable more extension into the channel of the main stream. This would be objectionable if navigation would be seriously impaired thereby.

The writer has a theory for the design, construction, and operation of such an ice and water separating and diverting works, but will not enter into many details for the present, as it would tend to distract the technical reader's attention from the main principles and facts established, and might be subject to some criticism by persons less accustomed to formal methods of thought and research. However, it may be permissible to explain briefly the operation of the sub-surface diversion channels and ridges.

It is evident that water must flow from the main stream to the divergent canal, if there is any draft by consumers along the canal. This will cause a greater or less tendency for all parts of the water in the main channel and diversion channels to flow toward the canal, as the water consumed or otherwise taken from the latter would naturally cause a fall of the water surface at the intake within the canal, and this would leave a surplus head or pressure of water at all parts in the main stream and diversion channels toward the intake.

The water in the main current, however, is flowing rapidly down the main channel because of the restricted cross-section in that direction, and the water in the transverse diversion channels is not, as it can have no component motion of any consequence at right angles

to the direction of the adjacent ridges. The water in the transverse oblique channels, between the ridges and below their crests, can flow with facility only in the direction of, or toward, the intake of the divergent canal, which it does.

Not so with the water at or near the surface of the main stream at places which are not below the crests of the ridges. This water does tend to move toward the intake, but only by reason of divergent or transverse accelerations which cause the main currents above the ridges to become more or less curved and concave toward the intake.

If the widths of the main stream and intake, the elevations of the crests of the ridges, the depths of the transverse channels, and the geographical configuration of earth and water in the vicinity of the intake have received proper attention with a view to design, alteration, construction, and operation, the surface currents of the main stream will not be impelled toward the intake sufficiently to cause any of the water at the surface to move into the intake and down the canal while it is at the surface.

It is a fact that the writer's method and invention put the forces indicated by this theory into actual operation and furnish a means for constructing and operating the intake of the canal and main channel in the vicinity of the intake, so that no surface water, and therefore no floatage, will enter the canal from the main stream. With sufficient intelligent application of the theory, the surface transportivity of the intake can actually be made negative, so that the surface of the water will flow outwardly therefrom into the main stream, the canal receiving a sufficient supply of water from the sub-surface or lower portions of the main stream to take care of the combined requirements of the canal and negative currents.

This effect can be produced with more or less intensity by making what may be called a sub-surface diversion of water to the canal from the lower portions of the main stream in greater quantity than would be necessary simply to supply the water required for the canal. The result of this is that the excess water diverted sub-superficially must cause a counter or compensating current flowing outwardly from the intake into the main stream.

Fig. 3 shows how the intake performs when ice is scattered all over the river. It may be remarked that the density of paraffin is only a trifle less than that of the heaviest ice, and about the same as

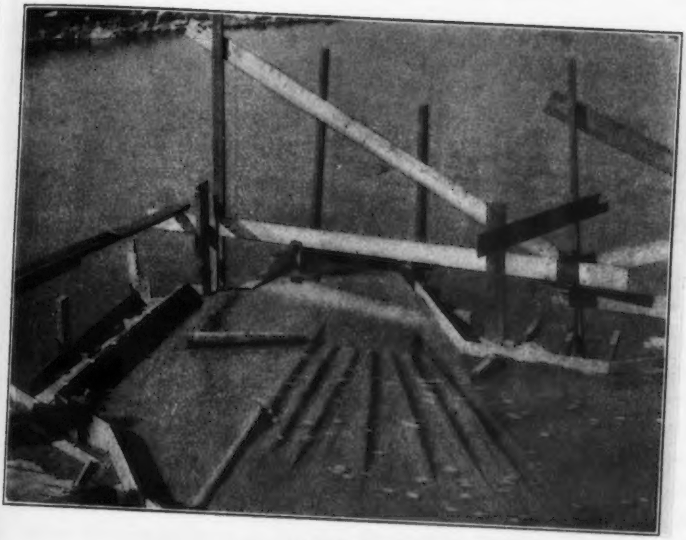

FIG. 3.

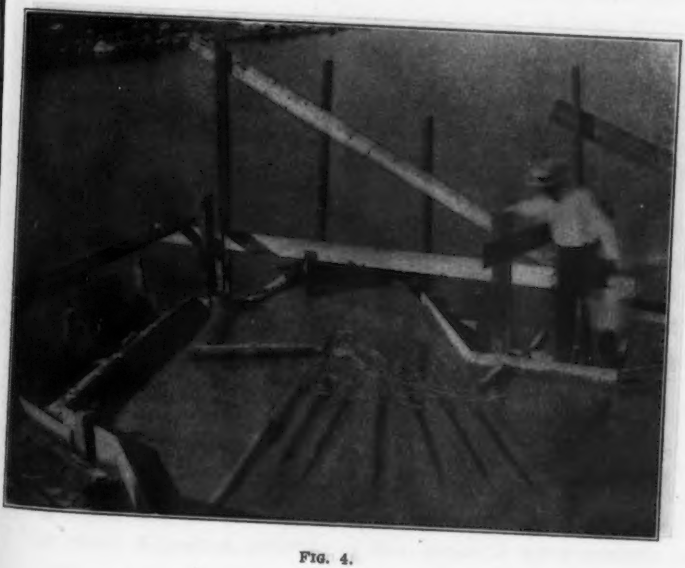

FIG. 4.

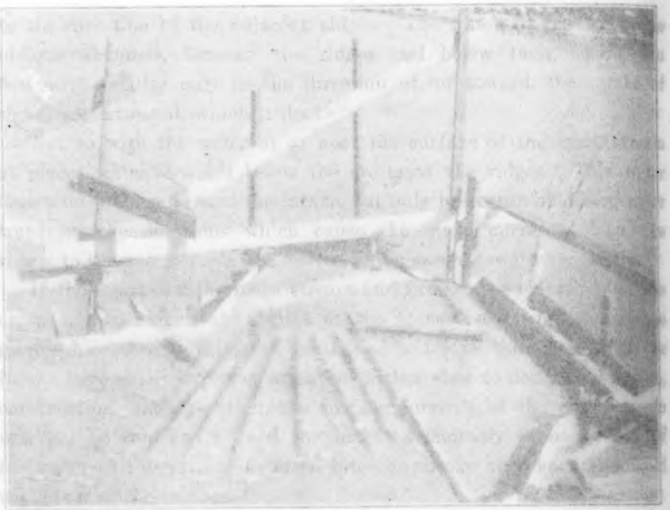

FIG. 3.

FIG. 4.

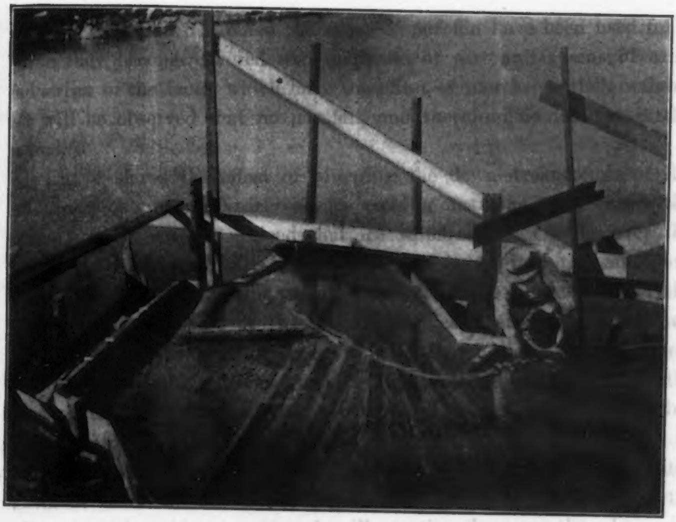

FIG. 5.

FIG. 6.

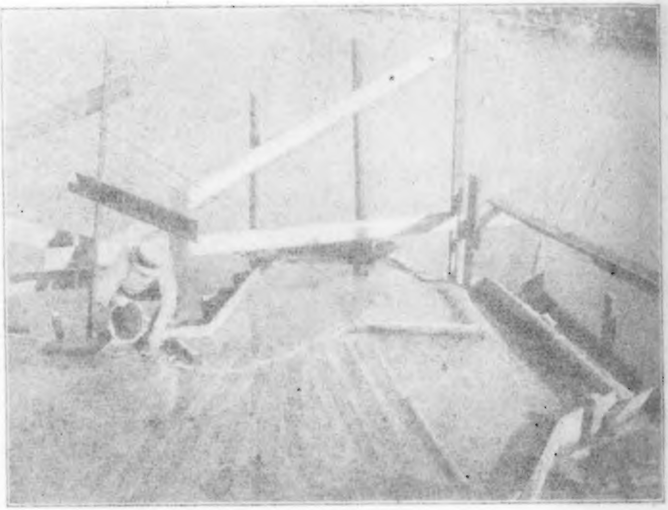

Fig. 5.

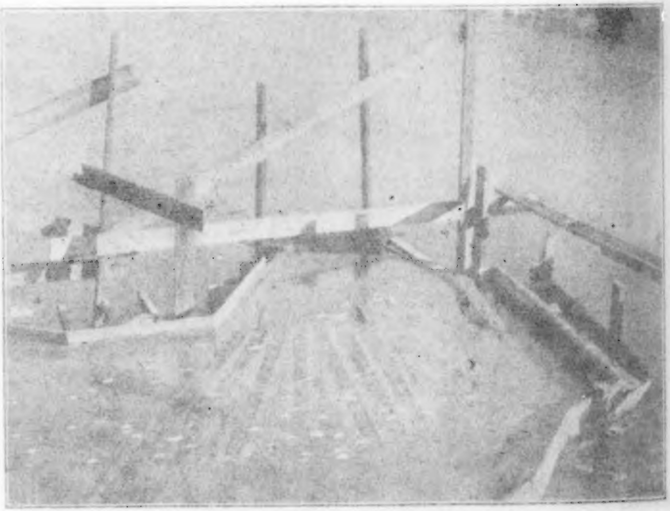

Fig. 6.

that of the lightest. When the cakes of paraffin have been used for a time they become covered with particles of dirt and grains of sand adhering to the faces, which have the effect of increasing their weight. It will be observed that no paraffin, and therefore no ice, enters the canal.

Fig. 4 shows the effect of plugging the down-stream ends of the diversion channels. Nearly all ice passing near the wing enters the canal, but it will be noticed that ice which chances to be over the unobstructed parts of the channels is not drawn into the canal but passes on down the main stream, away from the intake, beyond the influence of the canal draft. This is all clearly shown in the figure.

Fig. 5 shows the result obtained when the channels are completely obstructed by filling them with clay to the original elevation of the bed of the river. Here all the ice placed in the river at the wing passes into the canal.

Fig. 6 shows the operation of the intake in its natural condition, without diversion channels, ridges, or wings. The outlines of the diversion channels show in the illustration because the clay with which they are filled differs in color from the saw-dust concrete of which the bed of the river was constructed. A large proportion of river ice—more than half on the average—enters the canal.

Performance of Models.

The writer has experimented with small-scale hydraulic models several times. His conclusion concerning this matter is that models perform in much the same way as the full-size prototype. In fact, there was nothing in the results of the experiments to indicate that they did not perform exactly as their prototypes. These statements apply equally to hydraulic models of all kinds, whether they be of machines, such as water-wheels and pumps; of structures, such as overflow dams, weirs, and spillways; of sections of an actual river or canal; or of ships.

If, for example, a model of a section of a river has been constructed of sufficient size to prevent undue influences from properties of the fluid and materials which do not change by proper amounts with a change of scale, for example, viscosity, surface tension, etc., it will be found that the model performs almost exactly as the real section of river. Velocities, direction of flow, slopes of water surfaces, con-

figuration of eddies and bends, are all repeated in the model with great fidelity, supposing, of course, that the model has been accurately constructed and that we understand what is meant by mechanical similarity, as well as by geometrical similarity.

In the case of hydraulic models, it can be shown that homologous velocities in models of different size must be proportional to the square roots of homologous linear dimensions. When the quantities of water have been properly adjusted to comply with this requisite, it may be said that the mechanical and hydraulic conditions in the two models are mechanically and hydraulically similar, just as the configurations of fluids and solids in the models are geometrically similar.

This requirement of mechanical and hydraulic similarity has certainly been overlooked in some of the most important tests of models. Its neglect has led to the belief among many that models cannot be relied on, except as a means leading to rough approximations of doubtful value. Perhaps the tests of model water-wheels afford the most striking example. So far as the writer is aware, little heed has been taken to see that model water-wheels are tested under the proper heads; or, what is the same thing, if the head is fixed, as at Holyoke, that the size of the model wheel is properly proportioned to the head. This is saying nothing at all of the setting of the model water-wheel, which frequently bears no resemblance to the full-size setting.

Suppose, for example, that a water-wheel 110 in. in diameter is to be erected on a water fall of 180 ft. If the test is to be of a model operated under 16.5 ft. fall, then the diameter of the model should be only about 10 in., as the head for the model is only about the eleventh part of the total fall of water at the water fall. In short, the actual fall and that in the model must be in the same ratio as the linear dimensions of the homologous parts of the water-wheel and its model. It is this important requirement which is frequently lost sight of. As wheels of 30 in. or more are usually tested, it can be easily inferred that a test on such a size would result in too high a value for the efficiency in the case cited.

If the surfaces of the water passages in the model are made sufficiently smooth to represent correctly the homologous parts of the actual passages, the actual wheel and its model should perform alike. It can be shown that this simply imposes the condition that the resist-

ances to the flow of water over the homologous areas vary jointly as the homologous areas and the squares of the homologous velocities. This, we know, is nearly realized in practice. It follows that the foregoing statements are substantially correct.

The fact that homologous velocities must be proportional to the square roots of homologous linear dimensions is a fortunate matter, when the model is to be on a very small scale. Otherwise, the velocities in the model might be so small that they would not exceed Reynold's critical velocity, and thus the requirement of mechanical similarity would not be realized.

DISCUSSION

Mr. Leighton. M. O. LEIGHTON,* M. AM. SOC. C. E. (by letter).—The author is to be commended for his experimental work. Ice jams at power-plant intakes are of too common occurrence, and the interruptions to service or the necessity for supplemental power occasioned thereby result in large financial loss and inconvenience. The fact that so many men of experience differ so widely as to the best methods of handling ice is in testimony of the diverse conditions met in different localities. Therefore, identification of general principles that will apply either directly or in modified form to all conditions is much to be desired.

Mr. Groat's experimental principle is based on a separation of current direction between the upper and the lower parts of the stream section. By a series of submerged baffles he diverts the lower rather than the upper part of the stream into the diversion canal intake. By suitable modification of the channel he provides for an increase, or at least prevents a decrease, in the down-stream velocity of the upper portion of the moving water. The natural question that will occur to many readers is "What will happen to these transverse channels in the bottom of the main stream in the case of rivers that transport débris? Will they not be filled and their effectiveness thereby be eliminated?"

The foregoing question is very practical. The majority of streams in the United States carry more or less débris. Even in the regions of clear streams, like Northern New England and the far Northwest, a large aggregate quantity of material is rolled along on the bottom of stream channels. This material, being too coarse or too heavy to enter into suspension, is precisely the type that might be expected to lodge in these transverse channels and gradually fill them up. It is true that there might be some cases in which the lateral velocity generated in these transverse channels would keep the channels clear, but in such event Mr. Groat's device would become an efficient instrument for the diversion of large quantities of gravel, etc., into the diversion canal, which gravel, under normal conditions, would continue its course down stream. Thus, the canal operator would have to determine whether freedom from ice troubles for a few days in the year would be preferable to an all-the-year-round deposition of gravel and other material in his canal. Presumably, the losses and inconvenience arising from the former would eventually be far less than those from the latter.

The writer does not find that Mr. Groat has included this matter of débris in his discussion, and hopes he will do so in his closure.

* Washington, D. C.

Mr. Leighton.

It may be helpful to mention briefly the general conditions governing the transportation of débris in running water. Much work has been done, under the direction of the United States Geological Survey, at the hydraulic laboratory of the University of California, by Dr. Grove Karl Gilbert and Edward Charles Murphy, M. Am. Soc. C. E., the results of which have been published.*

A stream transports material in three ways:

(*a*) By Sliding or Rolling.—Débris so transported is that which is too coarse or too heavy to be raised by the current, but which is not heavy enough to resist the forward movement of the water;

(*b*) By Saltation.—Débris so transported moves by leaps or jumps. It is sufficiently light to be picked up by the current and carried for short distances—*i. e.*, from a few inches to a few feet—and then dropped on the bottom of the channel;

(*c*) Suspension.—Débris so transported is sufficiently light or fine to remain in suspension for longer periods. Some grades may drop to the bottom rather quickly when the current is checked, and some finer grades may remain in suspension almost indefinitely, even under quiescent conditions.

Of course, there is no sharp dividing line between the finer material that moves by suspension and the coarser material that moves by saltation. It is the material that is rolled along the bottom and that transported by saltation which might, and probably would, interfere with the complete success of Mr. Groat's device. Some idea of the complexity of stream traction and the difficulties encountered in devising any method for controlling the transportation of débris may be gleaned from the following paragraph, quoted from Dr. Gilbert's paper previously cited:

"The flow of a stream is a complex process, involving interactions which have thus far baffled mechanical analysis. Stream traction is not only a function of stream flow but itself adds a complication. Some realization of the complexity may be achieved by considering briefly certain of the conditions which modify the capacity of a stream to transport débris along its bed. Width is a factor; a broad channel carries more than a narrow one. Velocity is a factor; the quantity of débris carried varies greatly for small changes in the velocity along the bed. Bed velocity is affected by slope and also by depth, increasing with each factor; and depth is affected by discharge and also by slope. If there is diversity of velocity from place to place over the bed, more débris is carried than if the average velocity everywhere prevails, and the greater the diversity the greater the carrying power of the stream. Size of transported particles is a factor, a greater weight of fine débris being carried than of coarse. The density of débris is a factor, a low specific gravity being favorable. The shapes of particles affect traction, but the nature of this influence is

* *Professional Paper No. 86*, U. S. Geological Survey, under the authorship of Dr. Gilbert.

Mr. Leighton. not well understood. An important factor is found in form of channel, efficiency being affected by turns and curvature and also by the relation of depth to width. The friction between current and banks is a factor and therefore likewise the nature of the banks. So, too, is the viscosity of the water, a property varying with temperature and also with impurities, whether dissolved or suspended."

In the case of material rolled along the bottom or transported by saltation, it is manifest that grievous complications would arise in the application of Mr. Groat's device, and it can well be imagined, in view of the complexities cited in the paragraph just quoted, that no one can forecast precisely what would occur, at least in the present state of our knowledge of the subject. It is probable, too, that in very few situations would the actual events be similar in character. It appears to the writer, therefore, that this is one of the cases in which the performance of a model will differ materially from that of the full-sized prototype, unless the model is located so as to copy with exactness all the collateral conditions with respect to the transportation of débris. As it would be difficult, if not impossible, to copy such conditions on a model scale, it occurs to the writer that the first section of Mr. Groat's paper furnishes an excellent illustration of the probable fallacies in the last section.

Mr. Groat introduces a relatively new term, "transportivity", which he defines as the transporting capacity, of a river at a particular place, which is measured by the product of the width and the mean surface velocity. We have long been accustomed to use the term "capacity" to indicate the maximum load that a stream can carry, and though this term is ordinarily applied to the load in suspension, in saltation, or that rolled along the bottom, the analogy is so clear between these and the load that a stream can carry by flotation, that it seems unfortunate to introduce a new term. "Surface capacity" or "flotation capacity" would, in hydro-mechanical considerations of this kind, follow precedent, and be entirely definitive and satisfactory.

Mr. Smith. J. Waldo Smith,* M. Am. Soc. C. E.—It does not seem that the point made by Mr. Groat as to the efficiency and value of small models in solving hydraulic problems can be emphasized too strongly. It is truly surprising how nearly the results indicated by his models were verified by subsequent actual experience. The results obtained on small models are usually to be considered as being indicative rather than absolute, and this is especially the case in new fields of hydraulic construction where we have to deal with what may be called excessive head, quantity, or velocity. In such situations the search must be for a guiding experience which will tend to indicate all possible limitations

* New York City.

as to structural dimensions, and as to the factors of flow, of velocity, and of turbulence in the passing water. Mr. Smith.

With these ends in view, a model was constructed in connection with the design of the spillway and channel of the Boonton Dam, at Boonton, N. J. The line of the spillway channel was at a very acute angle with the axis of the overfall section, so that when the water had passed into the channel, after a fall of 50 ft., its velocity would have been very high. Such a condition would have called for a high and massive retaining wall, in order to keep the waters within the channel section. This problem was solved by constructing a model and by studying the behavior of the water when passing over it. As a result of these studies, a steep chute was constructed at one end of the spillway section, so that the water passing down it would acquire a high velocity parallel with the axis of the overfall, and thus by its weight and volume cause the resultant between it and the volume of the overflowing water to coincide with the axis of the channel.

At the present time, the general plans of the new overflow dam on the Schoharie development of the Catskill water system are being studied, and there will be built a good sized model of the dam and spillway before deciding on the various structural details involved. Here the dam is to be at right angles to the stream bed. The water will pass over its crest for a length of 1 300 ft., all on one side of the valley. The fall will vary from 160 ft. at one end, decreasing up the side of the valley to about 20 ft. The down-stream face of the dam is to be formed in large steps, from 10 to 20 ft. in height and from 10 to 18 ft. in width. The water will finally fall into a spillway channel along the toe of the dam, the bottom of the channel having a steep grade to the present stream bed. A high wing-wall will be built along the far side of the original stream bed, nearly at right angles to the dam and also at right angles to the axis of the spillway channel. It is expected that the model will give much information as to the action and behavior of the water and in determining many details as to the structural design, with particular reference to any features which may be needed in order to guide and control the direction of flow.

The author's reference to the skimming process recalls to the speaker's mind one of the best examples of this process which he has seen. At Omaha, Nebr., the very turbid Missouri River water was pumped into a settling reservoir, composed of a series of basins about 30 ft. deep. The water paused in these basins and then successively passed over long weirs separating the basins. The sediment settled quite rapidly, so that the thin skin passing over the last weir was, when seen, quite clear—for water of such character—and reasonably satisfactory for use without coagulation or filtration.

Mr. Fletcher.

ROBERT FLETCHER,* M. AM. SOC. C. E. (by letter).—The author has made a very positive and questionable statement on the "Performance of Models" in the following words:

"His conclusion concerning this matter is that models perform in much the same way as the full-size prototype. In fact, there was nothing in the results of the experiments to indicate that they did not perform exactly as their prototypes. These statements apply equally to hydraulic models of all kinds, whether they be of machines, such as water-wheels and pumps; of structures, such as overflow dams, weirs, and spillways; of sections of an actual river or canal; or of ships."

The inexorable laws of Mechanics forbid any such sweeping general conclusions. William M. Torrance, M. Am. Soc. C. E., has made a convincing exposition of this question in an article† entitled "Use of Models in Engineering Design," and has warned his readers against misconception of the value of such models in practical applications. Having made a model of a concrete water tank on a concrete trestle tower to a scale of 1:30 he showed that the model was proportionately stronger than its prototype of full size, inversely as the scale, or thirty times as strong. The apparently surprising strength and agility of diminutive creatures, like frogs, fleas, etc., in leaping power, was shown not to be remarkable, because the proportionate strength of their limbs, being as their cross-sections, are as the square of the linear scale of their size, though the weights are inversely as the cube of the scale; hence their relative muscular power is as the scale of their dimensions. Therefore, a frog or a flea can leap as far as a man can jump, because he has relatively more strength in proportion to his weight.

Commenting on this article, the writer added, among others, the following examples:‡

Passing from statical conditions to dynamical the following examples are to the point:

"Before the elevated railway was built in Boston the late Capt. J. V. Meigs made strenuous efforts to have his single-truss elevated railway adopted. The track or 'way' or truss was provided with two bearing rails for the horizontal wheels and two lower rails for the diagonal wheels of a unique form of truck, and the truss was supported on a single line of columns, as in the first New York elevated railway. This is fully described in Vol. VII, *Transactions*, Am. Soc. M. E., paper No. 189. An expensive working model was set up in a large upper room of a warehouse in East Cambridge, Mass., where the inventor demonstrated the 'successful working' of his scheme to visitors, as he did to the writer and a class of students in November, 1885.

* Hanover, N. H.

† *Engineering News*, December 18th, 1913.

‡ *Engineering News*, January 29th, 1914.

"The model engine weighed 275 lb., the tender 80 lb., with a model car six feet long. Steam was 'gotten up' quickly and the train developed an average speed of $10\frac{1}{2}$ ft. per second, passing curves of $6\frac{1}{4}$ ft. radius and mounting grades equivalent to 610 ft. per mile (adhesion increased by accumulator pressure on the horizontal truck wheels). On level track a speed of 22 ft. per sec. was made. Mr. Fletcher.

"This performance doubtless convinced the most skeptical; but on this occasion the spectators were reminded that the forces in action, especially the centrifugal reaction [not force, but the opposing effect of inertia as the body is incessantly pushed by the curving rail toward the center] on the curves, would be vastly out of proportion to the scale, in the full-sized train. The model was made on a one-eighth scale, and, if W represents the weight and v the velocity of the model, this centrifugal reaction would be expressed by $\frac{W}{g} \times \frac{v^2}{r}$. Assuming a velocity in the same proportion, one-eighth of, say, 66 ft. per sec. (45 mi. per hr.) $= 8\frac{1}{4}$ ft. per sec., and radius r of curve in like ratio, then, since W of the large engine will be as the cube of the scale, we have the action of the actual engine measured by $\frac{(8)^3\, W}{g} \times \frac{(8)^2\, v}{8\, r}$ $= 4\,096 \frac{W}{g} \times \frac{v^2}{r}$. To resist this the strength of track-frame, trusses, columns, etc., which depends upon the cross-sections, would be in the ratio of $(8)^2 = 64$ to 1: and the disproportionate dynamic effect of the full-sized engine would be $4\,096 \div 64 = 64$ times greater than any which the model could exert. The numerical measure of the larger effect would be, on a curve of $6\frac{1}{4} \times 8$ ft. radius: $4\,096 \times \frac{275}{32} \times \frac{8\frac{1}{4} \times 8\frac{1}{4}}{6\frac{1}{4}}$ $\risingdotseq 384\,000$ lb. This is 2.7 times the weight of the machine (275×8^3 $= 140\,800$ lb.); but it will be noticed that the radius is quite short and the speed (66 ft. per sec.) for so sharp a curve is not permissible. Right here is where the uninstructed experimenter is deceived if he infers proportionate dynamical stability from the beautiful behavior of the model.

"A different case is that of an experimental model dam. In a discussion we usually consider only a unit of length, say 1 ft. Assume a model 2 ft. high with any acceptable cross-section, to represent by a one-ninth scale a proposed dam 18 ft. high. Here relative *statical* effects depend upon the hydrostatic law that the [total] pressure varies as the square of the depth $\left(w \times h \times \frac{h}{2} = \frac{w\, h^2}{2} : w \times 9\, h \times \frac{9\, h}{2} = 81 \frac{w\, h^2}{2} \right)$. But the similar cross-sections also vary as the square of the scale; and all pressures and resultants in the two cases hold the same relative position. Hence, this is a case where the full-sized dam really has stability proportionate to that of the model.

"This condition, however, is far from true when overflow occurs. For instance, assume 6 in. gage depth of flow over the model, which would represent 54 in. over the proposed crest. Now the hydro-

Mr. Fletcher. dynamic law of discharge over weirs is (the quantity) $Q = c\ l\ h^{\frac{3}{2}}$, in which c is a constant, l the length and h the gage height above the crest; and $(9)^{\frac{3}{2}} = 27$. Here, then, the discharge over the full-sized dam would be 27 times that over the model; and, more than that, *it has nine times as far to fall* (really $9\frac{1}{2}$, from the center of gravity of the overfall). The energy acquired, which measures its capacity for destruction, varies with the height of fall and is, therefore, $9 \times 27 = 243$ times that of the water discharged over the model dam.

"Considering this inexorable mechanical law the designer may anticipate in some degree the tremendous impact against the back-water below, the scour on the bed of the channel, the 'kick-back' or reaction against the apron and toe of the dam, the vacuum effect upon the dam itself, etc.; but these would not be even faintly suggested by the behavior of the model and its 6-in. overflow."

It may be objected to this discussion that we have assumed 1 ft. in length, both for the actual dam and its model; whereas we should make the comparison by using only $\frac{1}{9}$ ft. of the model; and that, therefore, gives only 27 to 1; or, the relative effects are as the $\frac{3}{2}$ power of the scale ratio; but that is just where we are deceived. It is true that the total effect of shock and destruction along a line only one-ninth as long in the model as in the dam is only as 1 to 27; but the destructive effect will not be confined to a proportionate length. The convenient unit of 1 cu. ft. of water and 1 lin. ft. in both cases is proper, because it is required to compare the energy, symbolized by Wh (of the model), with that of $W'\ h'$ of the real conditions; that is, Wh compared to $27\ W\ 9\ h$; or, to state it otherwise, we are concerned here to compare the shock or destruction per foot or other equal length.

In a recent work,* J. L. Van Ornum, M. Am. Soc. C. E., gives an interesting account of some elaborate experiments made by German engineers at the Experimenting Establishment in Berlin. These are right to the point in this discussion. The problem and plan of procedure are stated thus:

"Because of uncertainties with regard to the attainment of entire success in being able to definitely represent on a small-scale model all the conditions of a natural river and to be assured that the effects of artificial modifications in the streams are correctly indicated by corresponding changes of the model, it was decided that the first requirement was the true reproduction of a definite part of the natural Weser River, and then to compare the effects of flowing water upon it with the state of the river bed which had been produced by corresponding conditions. If this experimental verification of the correctness of the details of reduction in scale and choice of materials proved satisfactory, then it could be confidently assumed that experimental

* "The Regulation of Rivers", pp. 163 to 172.

investigation of the effect of any proposed plan or detail of regulating works would show, on the model, the consequences that would result from the same construction of full size in the river itself. For this purpose a portion of the Weser, 1.6 km. in length * * * was chosen because of its availability in its definitely known characteristics both in its natural state and after works of improvement had been installed, the stability of its bed in showing similar conditions to exist at each recurring low water stage, and because a more effective regulation of that stretch of river was desired. Mr. Fletcher.

"The condition of the facilities of the hydraulic laboratories caused the adoption of a scale of linear reduction of 1 to 100 for both horizontal and vertical dimensions. With regard to the character of material which should be used for the bed of the artificial stream, it was evident that a variation in size corresponding to that of the river itself is important; but the question of the suitable proportionate size was not so clear. It was said that apparently the ratio of volumetric reduction should be as 1 to 100, which would call for an average diameter of grain of about 1.7 mm. inasmuch as that of the river gravel was about 8 mm. However, after some investigations of the behavior of graded sand of various average sizes under the action of flowing water, * * * a river sand of an average diameter of about 1.2 mm. was chosen for the material of the model; this was, in later experiments, changed to 1 mm., or six-tenths the diameter which would keep the volumetric ratio of the particles the same as the linear ratio of reduction for the general dimensions of the model."

So, to begin with, a scale reduction was made, reducing the size of particles to be transported to only 0.6 of that required by the geometrical relations. Then the account goes on to state that four other requirements had to be satisfied, *viz.*:

"A correspondence in relative height of mean low water, mean high water and mean water levels, the range in the model of course being one one-hundredth that observed in the Weser; all these water levels are to have a similar corresponding relation with respect to the tops of the groynes; the depths and cross-sections of the three water levels must have a like relation to those of the natural river; and the discharge in the model is to have a constant ratio to that of the Weser at all three stages mentioned".

It was considered that the continued product of the slope, depth, and reciprocal of the diameter of the grain should have the same value in both the model and the river. For the river this product was 0.0000235 and for the model 0.021; but we learn that:

"Repeated experimental attempts to attain satisfactory results on the basis of that computed slope seem to have proved disappointing; at any rate a slope of about one-tenth that value and a ratio of unit discharge of 1:40000 were experimentally found more satisfactory. Later, a surface slope of about 0.0015 and a corresponding discharge ratio of 1:50000 were found to produce a channel in the model still more nearly coincident with that of the river itself, especially for the higher stages."

Mr. Fletcher.

So here, again, preliminary trials led to very considerable changes in the computed quantities. Then, in conducting experiments, the bank protection and controlling groynes in the model were made with "small sacks of shot", and "small pieces of slate, at slopes of 1 on 1." It would appear that the density and stability of such materials are almost as much out of proportion to the resisting qualities of the materials on the actual river as would be steel sheet-piling *vs.* brush mattresses and rip-rap. Erosion in the model was prevented or corrected by these bags of shot.

Although some interesting similarities in effects were found, on comparing the profiles and cross-sections of the model and the river, it was admitted that there is

"A lack of coincidence in details which suggests the conclusion that the system is not yet perfected. Such irregularities are found as differences in distribution of shallow and deep portions of the channel, the smaller depths in the reach shown by the model, the greater variability in the experimental depths, and especially the greater comparative depths in the concave banks. While the last-mentioned difference is not important with regard to the effect upon navigability, it is, nevertheless, one of the characteristics by which the question of the adequacy of experimental methods must be judged."

Following the statement quoted at the opening of this discussion, Mr. Groat makes the following further positive statement:

"In the case of hydraulic models, it can be shown that homologous velocities in models of different size must be proportional to the square roots of homologous linear dimensions. When the quantities of water have been properly adjusted to comply with this requisite, it may be said that the mechanical and hydraulic conditions in the two models are mechanically and hydraulically similar, just as the configurations of fluids and solids in the models are geometrically similar."

He then states that experimenters with water-wheels have overlooked these relations and conditions, and have failed to make proper tests of model wheels.

Now the writer thinks that the instances cited by Mr. Torrance and himself plainly refute the idea of simple and always uniform interrelations and analogies. It is shown that, in a model of a static structure, additional load must be supplied to the extent of the weight of the model multiplied by the scale ratio. (A model bridge made by the writer had to be loaded with 350 lb. before the individual members were stressed in proportion to the homologous stresses in the bridge represented.)

In the single case of the model dam, as the pressure (load) itself varies as the square of the depth, the behavior of the model and its prototype are the same; but in all cases where dynamic effects are

involved, we cannot usually make the model and its prototype comparable or analogous by simple adjustments or contrivances. Mr. Fletcher.

Referring now to the experiments conducted by Mr. Groat, the writer would not presume to criticize the procedure or question the validity of the conclusions, so far as they relate to the particular object sought, and strictly under the conditions stated. No doubt, as the German experimenters found, useful lessons may be learned from "model" performances, but only under very special and tractable conditions. We have seen how their protracted and painstaking endeavors resulted in admitted failure to gain the full result sought. For one thing, changes in the model or miniature could be made with ease; but similar modifications under actual conditions might involve great labor and expense or develop unexpected forbidding conditions which the mere model would not suggest. (Like a plan "on paper" *vs.* a procedure necessitated in face of the working conditions.)

Engineers familiar with our northern rivers, even those flowing from north to south, and thus under more favorable conditions for getting rid of ice in the spring, know too well some of the extreme conditions that defy calculation. Although pieces of paraffin in a small stream may simulate ice carried under ordinary conditions of moderately high water, they are essentially different from ice. They will not freeze together as will ice after a thaw followed by a "cold snap"; they would not readily be subjected to the great side pressure which drives ice laterally into side channels and high up on sleeping banks; they would not so easily simulate the great jams which fill the entire channel, pile up high above it, and cause an excessive rise of the river, leading to destruction of dams, mill buildings, etc.; neither would the miniature contrivance be likely to produce baffling conditions of back-water which vex the souls of those who operate power plants. The writer's observations and experience on a river like the Upper Connecticut is that artificial furrows or transverse ridges in the bed of the river would be speedily obliterated in whole or in part, either by erosion or filling up; and such aids as jetties for controlling the flow, as proposed, must needs be of expensive construction to be permanent, and may easily be overtopped by high floods. The following instances illustrating the above stated points are only a few among many which might be cited.

At Sumner's Falls a rocky barrier extends so obliquely across the river that its length is nearly double the direct width of the river. About 50 years ago at this site there was a dam, a canal lock and approach canal for river boats, and a very large saw-mill running seven saws. A spring flood brought down ice which jammed and froze; a second flood increased the jam, piled the ice high on the dam and against the banks, and finally carried the mill down stream, and

Mr. Fletcher. wrecked the dam and lock, which were never rebuilt. The writer has seen* an impressive picture of blocks of ice up to 4 ft. thick (a man standing beside one) wedged together over an extent of many acres, on one side of the Lower Yellowstone dam, in Montana, as the result of a high flood. This suggests in part the possibilities of destruction by a spring flood carrying ice.

At the Vernon dam and power-house the spillway is 650 ft. long, and the fall, without flash-boards, is 32 ft. The river just below widens to 1 200 ft., but below that is a short curved narrows, about 400 ft. wide. Yet the engineer reports that in a high flood the water below the dam has risen to within 0.8 ft. of the crest, so as to make it for a time practically a submerged dam. How could any model dam and section of this bay and gorge, extending actually a mile on the concave, have suggested this condition of back-water?

When we consider the demonstrated fact that the transporting power of a stream varies as the sixth power of the velocity; that the energy of the flowing water varies as the cube of the velocity; and know that, by geometrical necessity, any model on a reduced scale lacks weight and stability in itself to test its full capacity, under diminutive conditions, we are obliged to object to the quoted all-inclusive claim for the validity of model studies and experiment, especially where hydrodynamic operations are involved.

Mr. Parker. A. F. Parker,† M. Am. Soc. C. E. (by letter).—The writer has been actively interested in the problem of canal intakes and keeping them clear. This paper mentions only the matter of ice and floating materials, and in large streams, presumably of not very great fall. Under conditions of ice flowing in such large rivers, and with only moderate velocity, the sub-diversion channels described may produce very good results; but, in smaller streams, of heavy fall, such as are usually found in mountainous districts, it is not so evident that the method presented would produce the results sought. In mountain streams it is usually necessary to build a diversion dam at each intake. Sometimes such dams may be permanent, and in other cases movable dams are necessary in order to pass the annual spring floods. In the case of a permanent dam, the basin back of it always fills up with silt, sand, and sometimes heavier drift materials, so that in time there is only a limited space of any considerable depth at the intake. Movable dams are erected only at low-water stages, and, when removed to pass the spring floods, the current sweeps the deposits accumulated in the basin cleanly away. Thus the action resulting from the use of either form of dam would evidently preclude the use of sub-diversion channels.

In such cases—and such conditions obtain almost everywhere in mountainous localities—it is always very difficult to keep intakes clear

* *Engineering News*, November 14th, 1912, p. 898.
† Ogden, Utah.

Mr. Parker.

of ice. The main reliance must be placed on drawing the water from the greatest depth possible below the surface. The still water above the dam holds the ice back, but sometimes the mush ice reaches nearly the full depth of the water. Usually a large gang of men is required to keep the intake clear, and it is impossible to prevent considerable quantities of ice from entering it.

In streams of the kind mentioned, however, the ice problem is the [illegible] involved [illegible] makes [illegible] difficult to contend with sand, gravel, and floating and [illegible] driftwood. During high water driftwood lodges against trash racks [illegible] commonly placed [illegible] materials [illegible] then, carried sand, [illegible] and silt, often [illegible] almost water-tight dams; sometimes it is difficult to keep such racks clear, and then there are formed sand and gravel [illegible] in canals and thence down the canal [illegible] pipe line, cause injury to water-wheels, or [illegible] and are not easily [illegible]. It is evident that such [illegible] likely to be easily [illegible] by the river drift [illegible] of high water, even in large slowly moving streams; it is absolutely sure that [illegible] flowing streams [illegible] to the mountains would either fill them up or that they would serve to divert the sand and gravel—carried in great quantities by the water—[illegible] it is desirable to avoid [illegible] conditions differing so much from those assumed [illegible] streams, [illegible] excuse is in the great interest [illegible] importance of the matter under [illegible] conditions. The writer submits [illegible] (Fig. 7) showing a [illegible] designed for an irrigation and power canal, where it was very [illegible] to deal with the winter and the floating drift, sand, and gravel in summer. The intake was not built just as shown on the plans, and does not now give the best results. Had it been built as designed, it would surely have done the work in the best possible manner.

The plans show a movable dam across the stream, operated from a service bridge. [illegible] in place of the usual trash rack, there is a vertical diaphragm [illegible] fastened [illegible] beams extending [illegible] under concrete [illegible] and reaching in a straight line up and down stream and nearly parallel to it. This diaphragm is set above the [illegible] of a concrete [illegible] with side-walls [illegible] with the bottom of the stream. The [illegible] is discharged below the dam, the flow through the concrete channel being controlled by a wooden gate. The diaphragm wards off all floating materials, and the space under it allows the water to pass into the intake. The channel or trough of concrete catches all sand and gravel passed down by the current, and the rush of the current of water through it removes all such material and prevents clogging; the channel has been given a heavy fall in order to accomplish this.

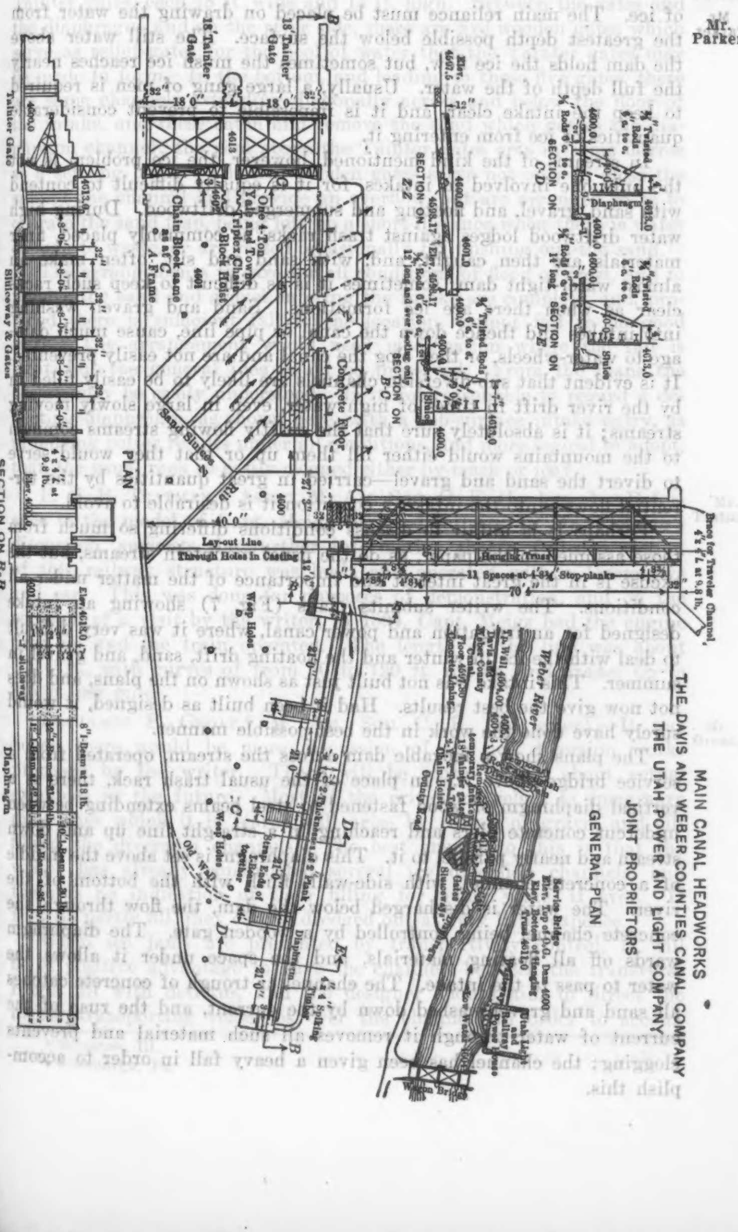

FIG. 7.

Mr. Parker. of ice. The main reliance must be placed on drawing the water from the greatest depth possible below the surface. The still water above the dam holds the ice flow, but sometimes the mush ice reaches nearly the full depth of the water. Usually, a large gang of men is required to keep the intake clear, and it is impossible to prevent considerable quantities of ice from entering it.

In streams of the kind mentioned, however, the ice problem is not the only one involved in intakes, for it is equally difficult to contend with sand, gravel, and floating and submerged driftwood. During high water driftwood lodges against trash racks, as commonly placed, finer materials are then caught, and, with sand and silt, often make an almost water-tight dam; sometimes it is as difficult to keep such racks clear as when there are ice formations. Sand and gravel, washing into intakes and thence down the canal or pipe line, cause much damage to water-wheels, or they clog the canal and are not easily prevented. It is evident that sub-diversion channels are likely to be easily filled in by the river drift in times of high water, even in large slowly moving streams; it is absolutely sure that the swiftly flowing streams common to the mountains would either fill them up or that they would serve to divert the sand and gravel—carried in great quantities by the torrential flows—into the intake, a condition it is desirable to avoid.

Perhaps it is unfair to discuss conditions differing so much from those assumed in the paper, as do the usual mountain streams, but the excuse is in the great interest and importance of the matter under all conditions. The writer submits plans (Fig. 7) showing an intake designed for an irrigation and power canal, where it was very difficult to deal with the ice in winter and the floating drift, sand, and gravel in summer. This intake was not built just as shown on the plans, and does not now give the best results. Had it been built as designed, it would surely have done the work in the best possible manner.

The plans show a movable dam across the stream, operated from a service bridge above it. In place of the usual trash rack, there is a vertical diaphragm of wood fastened to steel beams extending between undercut concrete piers and reaching in a straight line up and down stream and nearly parallel to it. This diaphragm is set above the middle of a concrete channel, with side-walls flush with the bottom of the river. The water is discharged below the dam, the flow through the concrete channel being controlled by a wooden gate. The diaphragm wards off all floating materials, and the space under it allows the water to pass to the intake. The channel or trough of concrete catches all sand and gravel washed down by the current, and the rush of the current of water through it removes all such material and prevents clogging; the channel has been given a heavy fall in order to accomplish this.

Mr. Parker.

Below the diaphragm, and some distance away, there are two Tainter gates, each 18 ft. wide and 13 ft. high. Between the gates and the diaphragm, and on the river side, there are five wooden gates which serve as relief gates for the pool of water that the Tainter gates may be made to form. In the bottom, and leading to these five gates, there are some channels extending diagonally across and below the floor of the intake, and these catch and remove the sand which passes the diaphragm channel; the pool above the Tainter gates acts in some degree as a settling basin. The five wooden gates serve not only to close the side of the channel and provide an overflow, but are arranged so as to operate the sand trap, being left up a short distance when there is water enough to waste, or are raised occasionally for flushing out the sand.

The arrangements to care for all conditions of flood or frost, and to remove the sand and gravel entering the intake are complete, and, provided there is sufficient fall to the location, cannot fail to do good work. The arrangement would need modification where the fall is light, but the general idea could be preserved. From the plans the design may be easily understood. Is it not reasonable to regard a deflecting diaphragm, as shown here, with a sub-surface intake entry, as promising better results under all conditions, and more logical to adopt, than the usual rack so easily clogged either by trash or ice?

Mr. Tratman.

E. E. R. TRATMAN,* ASSOC. M. AM. SOC. C. E. (by letter).—Reference to the model of Capt. Meigs' elevated railway, in Mr. Fletcher's discussion, suggests the further information that a full-size section of this railway structure was built and operated with a locomotive and car. This was done for purposes of demonstration, and on the occasion of a visit by the writer, in 1894, Capt. Meigs had the engine fired up and the train operated. The length of the line was about 1 114 ft., with curves of from 50 to 120 ft. radius, and grades up to 345 ft. per mile.

Mr. Groat.

BENJAMIN F. GROAT,† M. AM. SOC. C. E. (by letter).—It was hoped there would be more discussion of the ice diversion works proposed by the writer. There will be more to say about actual examples in the future. Most of the criticisms relate to imaginary difficulties which it is feared will arise by reason of gravel and silt filling the diversion channels. The best answer to this is that the writer has already constructed several ice diversion channels, and there is no indication of their filling for many years, if ever. If they do partly fill, they can be dredged out at very little expense.

When ice jams are prevented by transportivity control, it will be of positive advantage to have the channels between the transverse ridges fill with detritus, and the designer should aim to direct the higher transporting velocities over the channels in order to accom-

* Wheaton, Ill.

† Pittsburgh, Pa.

Mr. Groat. plish this result. In such cases the transverse channels will have been left empty simply to save the expense of filling them.

No comprehending designer would plan channels causing velocities capable of throwing detritus into an intake. Bottom transporting capacity can be controlled, as well as transportivity. The proper design first separates and preserves the lower currents of water, containing depositable material, from the upper currents of surface water and floatage, after which the latter are diverted down stream with the lower currents of detritus.

The successful operation of ice-diversion works depends not alone on channels, but also on an intelligent general planning and location of the structures.

Mr. Parker's discussion is valuable and welcome. No doubt it is difficult to deal with mountain torrents, and the question of detritus and submerged timber is serious at times, even on the larger rivers of low velocity. In the latter case it is known that bad winters frequently bring to certain plants damage amounting to several hundred thousand dollars, due to such causes. If the racks are taken out, stones get into the turbines; if the racks are kept in place, they plug with ice and cause a shut-down.

The writer is certain that rack spacing has been generally too small, and that, with the rapidly increasing sizes of turbines, there should be a commensurate increase in size of rack openings, the design of which should take into account very carefully the sizes and character of timber and floatage actually occurring in the feed water.

Records should be kept at the power-house to determine these questions, and thus there may be designed a more appropriate screen. Records of this kind are not usually available before the power-house is built. Racks are not the only element of a power-house which may need to be reconstructed after the plant has been in operation for a time.

The writer is now designing a screen for a large power-house on the principle that it should be a unit distinct from the power-house, rather than be apportioned in individual screens among the turbine units. The advantage of this is that, if a portion of the screen is plugged, it will not be necessary to shut down any turbine, as, by the plan mentioned, water from any part of the screen will feed to any and all wheels. Moreover, individual screens are illogical when considered financially. The investor is not only compelled to pay a large sum to secure a sufficient cross-section for a water passage in concrete leading to a turbine, but is also compelled partly to fill it again with an expensive steel obstruction throughout the section. In many situations, as Mr. Parker suggests, racks may be omitted, at least for part of the year.

Mr. Groat.

Mr. Parker closes by asking the question:

"Is it not reasonable to regard a deflecting diaphragm, as shown here, with a subsurface intake entry, as promising better results under all conditions, and more logical to adopt, than the usual rack so easily clogged either by trash or ice?"

The writer replies to this by saying that he prefers not to capture gravel, but rather to divert it in the manner indicated in the paper; that he is well acquainted with several deflecting diaphragms, like Mr. Parker's in principle, and that, so far as diverting ice is concerned, they can be considered as nothing short of colossal failures—colossal because of their excessive cost.

The writer does not doubt that such a diaphragm might operate successfully in deep water when the draft is relatively small, but prefers the open intake, with sub-surface diversion channels and ridges to throw the ice away from the intake and divert it down the main channel. In the case of intakes, much more than with screens, the investor pays a large sum to secure a sufficient waterway to his power-house, and then engineers proceed to fill it with expensive concrete piers, steel frames, racks, and perhaps diaphragms, the latter taking up more cross-sectional area that could otherwise be made available than all the rest of the redundant structures referred to. A diaphragm cannot produce a real sub-surface diversion.

The writer thinks that the gravel trap under the diaphragm, as described by Mr. Parker, is unnecessary. Its action cannot depend on the heavy longitudinal fall given its bottom, but only on the slope of the water surface along the streamward face of the diaphragm. At least, this must be so when there is much depth of water in the pond. Thus, the rush of water out of the gravel trap through the wooden gate would be confined to the lower end of the sluice, thereby allowing gravel to accumulate at the middle and upper end.

It would be better to eliminate the sluice, and provide a complementary diaphragm immediately under the upper one and extending upward from the stream bed sufficiently to cut off the flow of gravel to the opening between the two diaphragms, depending on the current of the stream, aided by the wooden gate when necessary, to wash the gravel down stream. The writer once actually secured a small supply of water free from floatage and detritus by just such a contrivance.

Professor Fletcher makes the mistake of considering the immensity of the consequences of an ice jam after it has formed. These are the things which ice diversion ridges and channels will prevent, not by taking up an ice jam bodily and shaking it to pieces, but by preventing the accumulation of the pieces.

Mr. Groat.

Ice jams can surely be prevented by transportivity control at critical places, in such streams as the Niagara and St. Lawrence, when the necessary expenditures can be justified.

The photographs of the model (Figs. 8 and 9) illustrate the effects of transportivity control in a situation different from that represented by the model shown by Figs. 1 to 6. If the transverse ridges were carried entirely across the main stream, and were properly designed, they would prevent an ice jam in that location, supposing jams to be likely to occur there.

Models.—The nature of the discussion discloses the main facts which the paper intended to emphasize, namely, that the possibilities in ice diversion and in designing by using hydraulic models are not fully appreciated by engineers generally, and that such demonstrations as the paper affords are imperatively needed.

This is illustrated forcibly by Mr. Leighton's quotations and remarks which are closed by him as follows:

"As it would be difficult, if not impossible, to copy such conditions on a model scale, it occurs to the writer that the first section of Mr. Groat's paper furnishes an excellent illustration of the probable fallacies in the last section."

Professor Fletcher closes his remarks by saying:

"When we consider the demonstrated fact that the transporting power of a stream varies as the sixth power of the velocity; that the energy of the flowing water varies as the cube of velocity; and know that, by geometrical necessity, any model on a reduced scale lacks weight and stability in itself to test its full capacity, under diminutive conditions, we are obliged to object to the quoted all-inclusive claim for the validity of model studies and experiment, especially where hydro-dynamic operations are involved."

In fundamental contrast with such ideas, there are the valuable results of actual experiments with models as used by Mr. Smith in designing the spillway and channel of the Boonton Dam. It is to be hoped that Mr. Smith will soon publish the results of the model tests which he is now conducting relative to the plans for the Schoharie development of the Catskill water system, and, later, the results of observations on the actual structure for comparison.

Mr. Hunt's suggestion during the oral discussion is also quite enlightening; that is, a model is the only means for reproducing all the complexities of flow caused by natural channel conditions. The writer replied to this by saying that there were other methods, but now wishes to qualify this by stating that he believes Mr. Hunt is about right; at least, that the model is by far the best means.

The writer purposes to show that hydraulic models can be, and have been, constructed and operated so as to reproduce with accuracy

FIG. 8.—MODEL, SHOWING EFFECT OF TRANSPORTIVITY CONTROL.

FIG. 9.—MODEL, SHOWING EFFECT OF TRANSPORTIVITY CONTROL.

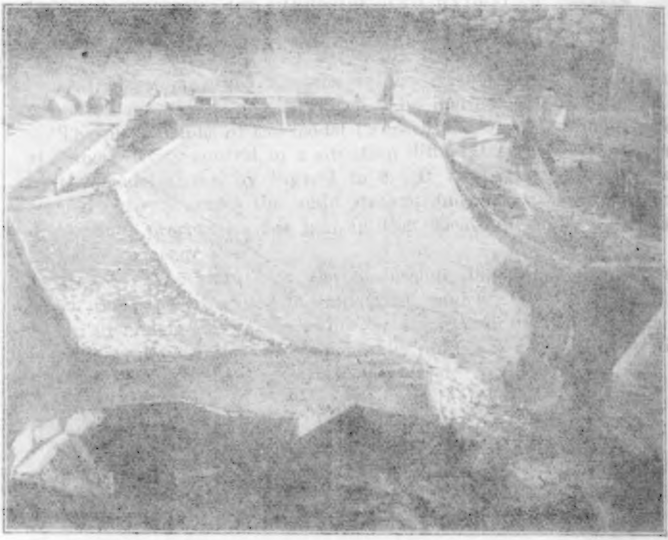

Fig. 8.—Model Showing Effect of Transportivity Control.

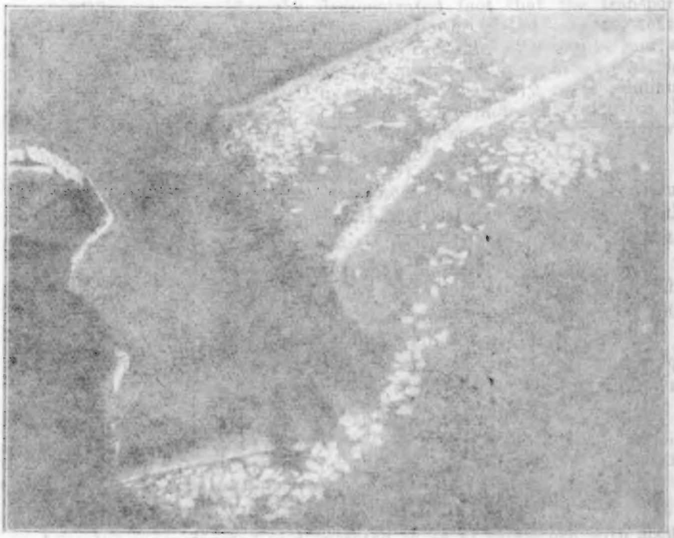

Fig. 9.—Model Showing Effect of Transportivity Control.

the performances of the corresponding full-size prototypes. This is especially true of the conditions of flow, and, when this is once grasped by hydraulic engineers, there will soon be no further lack of experimental data. Mr. Groat.

It will be demonstrated that, when a hydraulic model is constructed and tested under contemporaneous conditions of geometric and dynamic similarity, the reactions and velocities in the model are the relative counterparts of the corresponding reactions in the prototype, and that the counterparts include:

(1) Extraneous forces, such as gravitational forces, and centripetal forces or centrifugal reactions, when the latter are produced by the rotation of the model as a whole;
(2) Internal, centripetal forces, or centrifugal reactions; that is, those due to reactions between different masses in the model, as in an eddy, for example. This also includes tangential accelerations which, of course, are of the same dimensions as centripetal accelerations;
(3) Static and dynamic effects due to the necessarily imposed proportionality of heads and depths;
(4) Skin friction;
(5) Effects of wind, more particularly as regards its effect on the course of running ice;
(6) Transportation of detritus;
(7) Transportivity of the water for floatage;
(8) Time for the model is made consistent by reducing the period of time corresponding to observations on the prototype in the ratio of the square root of linear dimensions; if the unit of time is reduced in that ratio, a watch may be made for the model so that the time indicated by this watch will be numerically equal to that of the corresponding observation on the prototype taken by an ordinary watch; thus, time in the model becomes the counterpart of that actually;
(9) Measurements; besides time, scales of length, area, volume, weight, force, etc., may be based on the linear scale ratio, so that measurements of all kinds, including time, as mentioned above, may be made directly on the model, thus avoiding calculations to ascertain what the corresponding quantities should be for the prototype;
(10) Factors of safety in structures, provided the models are properly constructed and properly tested.

The question of viscosity, from a theoretical point of view, is less satisfactory, but the practical circumstances under which viscosity is operative are such that it may be treated with skin friction, on the theory that all hydraulic resistances finally result in dynamic reac-

Mr. Groat. tions converting kinetic energy into heat, through the medium of viscosity and kindred resistances.

However, it is certainly questionable to consider skin friction as due to viscosity unless a much narrower definition is given to skin friction than ordinarily. It is better to consider all obstructions to flow as offering direct resistances to the water, the translatory kinetic energy of which is immediately reduced in the direction of flow, the difference being converted into kinetic energy of turbulence and eddies, ultimately to be absorbed by viscosity friction.

Surface tension may become very troublesome if the model is not large enough. This subject needs much investigation. Surface tension has a preponderating influence on weirs when the head is very slight.

Although the intention is to confine the study to the theory of hydraulic models, much of what is said will apply directly to other types—model structures, for example—and thus an opportunity will be provided for investigating some of the alleged misdemeanors of models.

Theory of Hydraulic Models.—Several writers have given very brief and incomplete accounts of the theory of models, among which may be mentioned:

Routh, on "Elementary Rigid Dynamics" (1897), Articles 367 to 374, inclusive. These paragraphs relate to the principle of similitude in models and in other very interesting cases.

Stanton and Pannell, in a paper entitled "Similarity of Motion in Relation to the Surface Friction of Fluids." This is a British publication from the Collected Researches of the National Physical Laboratory, which paper every hydraulic engineer should study. The word "similarity" in the title of this paper is used in a somewhat different sense from that adopted in the present discussion.

The writer, in *Engineering Record,* September 25th, 1915, page 377.

Forchheimer, on "Hydraulik", page 33.

Buckingham, in *Transactions,* Am. Soc. Mech. Engrs., Vol. XVII (1915), on "Model Experiments and the Forms of Empirical Equations."

Durand, on Model Propellers, in Report 14 to the National Advisory Committee on Aeronautics, Washington.

The writer, in *Canadian Engineer,* February 14th, 1918, page 139.

Van Ornum, in "The Regulation of Rivers", devotes several pages to an interesting account of actual tests on models of portions of rivers.

Mr. Groat.

The ideal way to prove the equivalency of models and their prototypes would be to determine the performance of each by the theories of hydrodynamics, and then compare the results, homologue with homologue. The difficulties of integrating the partial differential equations for the simplest cases are so great, however, that the method must be abandoned for lack of time and perspicuity, if for no other reason.

Perhaps the first elementary step taken toward proving the principle of hydraulic similarity should be by way of the Chezy formula

$$v = c\sqrt{r\,s} \qquad \text{(1)}$$

If we have a model of an actual river channel, and represent its elements by small Italic letters, allowing Equation (1) to indicate the relations for the model, then the corresponding elements for the river (prototype) will be represented by the capital Italics, and we shall have,

$$V = C\sqrt{R\,S} \qquad \text{(2)}$$

Squaring and taking ratios,

$$\frac{\dfrac{v^2}{r}}{\dfrac{V^2}{R}} = \left(\frac{c}{C}\right)^2 \cdot \frac{s}{S} \qquad \text{(3)}$$

If Roman capital letters now be understood to represent the ratios of corresponding small Italics to corresponding capital Italics, as $\mathrm{V} = \dfrac{v}{V}$,

$$\frac{\mathrm{V}^2}{\mathrm{R}} = \mathrm{C}^2\,\mathrm{S} \qquad \text{(4)}$$

Now the model is geometrically similar to the prototype, so that, if we are to have hydraulic similarity, $\mathrm{S} = 1$. If the surface of the model is geometrically similar to that of the prototype, as supposed, we should also have $\mathrm{C} = 1$. Therefore,

$$\left.\begin{aligned} \frac{\mathrm{V}^2}{\mathrm{R}} &= 1 \\ \text{or,}\quad \frac{v^2}{r} &= \frac{V^2}{R} \end{aligned}\right\} \qquad \text{(5)}$$

Equation (5) shows that the velocities should be proportional to the square roots of the hydraulic radii, and, therefore, to the square roots of any homologous linear dimensions, by reason of geometric and hydraulic similarity, which latter extends to heads and depths as well as to any other linear wet dimensions. If the velocities and linear dimensions are not thus connected by Equation (5), the

Mr. Groat. hydraulic slopes will not be equal, thus causing the similarity of flow to vanish.

Table 1,* giving Chezy coefficients, is unique, and shows at a glance that the value of c, as taken from the table, will be approximately constant when some attempt is made to select values in such a manner as to maintain a constant ratio between the probable dimensions of the irregularities of the surfaces and the hydraulic radius. The writer has tried this several times, with good results, but, to avoid any possible prejudice, invited a young man familiar with such matters to try it, thereby securing the results shown in Table 2, which need no explanation except to state that the average of the values of the Chezy coefficient for the larger and smaller of the radii compare as follows: $c = 106\frac{1}{8}$; $C = 104\frac{3}{8}$. This is a practical demonstration of the accuracy of model performance as representative of the flow in an actual channel. The only obstacle to more accurate work is the uncertainty as to the magnitudes of the irregularities of the surfaces, both actual and as indicated in the table.

The simple demonstration just given connects only the mean velocity, longitudinal surface profile of the water, mean depth, and irregularities of wet surface at each of the individual cross-sections of the model with the homologues in the prototype. It does prove, for example, that the hydraulic gradient is repeated to scale in the

TABLE 1.—Values of c Deduced from Darcy and Bazin's Values.

Hydraulic mean depth = m.	Very smooth channels. Cement.	Smooth channels. Ashlar or brickwork.	Rough channels. Rubble masonry.	Very rough channels. Canals in earth.	Excessively rough channels encumbered with detritus.	Hydraulic mean depth = m.	Very smooth channels. Cement.	Smooth channels. Ashlar or brickwork.	Rough channels. Rubble masonry.	Very rough channels. Canals in earth.	Excessively rough channels encumbered with detritus.
0.25	125	95	57	26	18.5	8.5	147	130	112	89	....
0.5	135	110	72	36	25.6	9.0	147	130	112	90	71
0.75	139	116	81	42	30.8	9.5	147	130	112	90	....
1.0	141	119	87	48	34.9	10.0	147	130	112	91	72
1.5	143	122	94	56	41.2	11	147	130	113	92	....
2.0	144	124	98	62	46.0	12	147	130	113	93	74
2.5	145	126	101	67	....	13	147	130	113	94	....
3.0	145	126	104	70	53	14	147	130	113	95	....
3.5	146	127	105	73	....	15	147	130	114	96	77
4 0	146	128	106	76	58	16	147	130	114	97	....
4.5	146	128	107	78	....	17	147	130	114	97	....
5.0	146	128	108	80	62	18	147	130	114	98	....
5.5	146	129	109	82	....	20	147	131	114	98	80
6.0	147	129	110	84	65	25	148	131	115	100	....
6.5	147	129	110	85	....	30	148	131	115	102	83
7.0	147	129	110	86	67	40	148	131	116	103	85
7.5	147	129	111	87	....	50	148	131	116	104	86
8.0	147	130	111	88	69	∞	148	131	117	108	91

* Copied from Professor Unwin's article on "Hydraulics" in "The Encyclopedia Britannica", Vol. 14, p. 69, Eleventh Edition.

Mr. Groat.

TABLE 2.—COMPARISON OF CHEZY COEFFICIENTS FROM UNWIN'S TABLE IN "ENCYCLOPEDIA BRITANNICA."

Nature of channel.	Irregularities.	Ratio.	Hydraulic radius.	C. from Table.	Hydraulic radius.	C, from Table.
Smooth channels, cement.	1/64 in.	800%	0.25	125	2.0	144
Smooth channels, brick...	1/8 in.		2.00	124	16.0	130
Smooth channels, brick...	1/8 in.	1 600%	2.00	124	0.5	110
Rubble channels..........	2 in.		32.00	115	8.0	111
Rubble channels..........	2 in.	1 200%	4.0	106	1.0	87
Rough canal in earth.....	2 ft.		4.8	104	12.0	93
Rough canal in earth.....	2 ft.	500%	10.00	91	2.0	62
Very rough earth with detritus..................	10 ft.		50.00	86	10.0	72
Sum of smoother..				446	....	403
Sum of rougher..				429	....	406
Average of eight values, smoother.....................				106⅛	....	....
" " " " rougher..............................				104⅝	....	....

model, and that the wet surfaces of the model must be made geometrically similar to their homologues in every respect. It is, indeed, the virtual integrations "in parallel" of the differential equations for the model and prototype, making their hydraulic slopes (derivatives) equal, and, like integration, it requires the supposition of the existence of homologous depths and velocities, and equality of hydraulic slopes to begin and end with; that is, there must be limits for the integration.

This similarity suggests a practical fact which must be heeded very carefully when a model is to be tested. The conditions of flow surrounding the entrance and discharge of the water to and from the model must be properly adjusted, to represent exactly those existing in the prototype, or there will be no establishment of hydraulic similarity. When the prototype is merely a proposed construction, this part of the test must be determined by the most thorough investigation. When the prototype exists, the conditions of entrance and discharge must be reproduced exactly. This is confirmed by an extensive experience with the hydraulic models illustrated photographically in this paper.

The writer might proceed further with the interpretation of results, and show that any two homologous forces in model and prototype are bound to one fixed ratio. The skin frictions on homologous areas, for example, are in the same ratio as the weights of homologous volumes, or as the pressures of water on homologous areas. Demonstrations of such dynamical similarity, however, will be allowed to develop in the sequel.

The most serious objection to Equations (4) and (5) is that they do not extend to details of the flow, such as the distribution and directions of velocities, eddies, boilers, and other local disturbances.

Mr. Groat. That these, too, are reproduced in the model, will develop, either directly or as collateral results.

Another difficulty undoubtedly lies in the fact that the flow in a river is not steady, the particular configuration existing at one instant probably never again being exactly repeated. A rigid demonstration would draw on the theory of statistics to determine an average, as compared with an instantaneous configuration. Fortunately, actual experiments are of a statistical nature, especially if the effort is to determine average values rather than to record isolated instantaneous observations.

Verification.—Optical demonstration is very convincing. The map, Fig. 10, shows the results of actual comparison of model and prototype.* It represents a section of the St. Lawrence River where a current impinges on the head of an island at *N*, being thereby diverted partly into each of the two channels forming the island.

The quantity of water supplied the model was adjusted so that the mean velocity in the model would be proportional to the square root (1:10) of the scale ratio, 1:100. Approximate adjustments were made at the inlet and outlet of the model to secure proper entrance and discharge conditions as regards distribution of velocities and surface levels. The paths and velocities of floats, and the levels of the water surface at various points were then compared with the homologues observed in the actual river by means of the plottings shown on the map. The model was tested in 1917, whereas the observations on the prototype were those made in 1904 by John R. Freeman, M. Am. Soc. C. E. The hydrography of the river did not change in the meantime, owing to the stable nature of the channel.

The discharge in the model was for a stage of 3.8 at Gauge *A*, which was exactly the stage of the river when the actual surface elevations were observed as indicated. An extremely close agreement between individual pairs of homologous elevations need not be expected, because the elevations in the model were determined rather roughly by an ordinary rule held vertically on the heads of submerged nails serving as bench-marks. The rule was graduated to hundredths of a foot to conform to the scale ratio of 1:100. A more perfect check on the correspondence of water surface levels can be made statistically by averaging all the level readings, having some regard for weights, as in Table 3.

The discrepancies of elevation being pretty well distributed between plus and minus, and the average elevations agreeing closely, it may be concluded that the contours of the water surface were well represented to scale in the model; moreover, the difference in level between *M*

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* The writer is indebted to Messrs. F. W. Ely and E. O. Garrett for much assistance in attending to the matter of constructing and testing the models and, at a much earlier date, to Mr. D. C. Walser for corresponding work in connection with a model built to a scale of 1:1 000.

Mr. Groat.

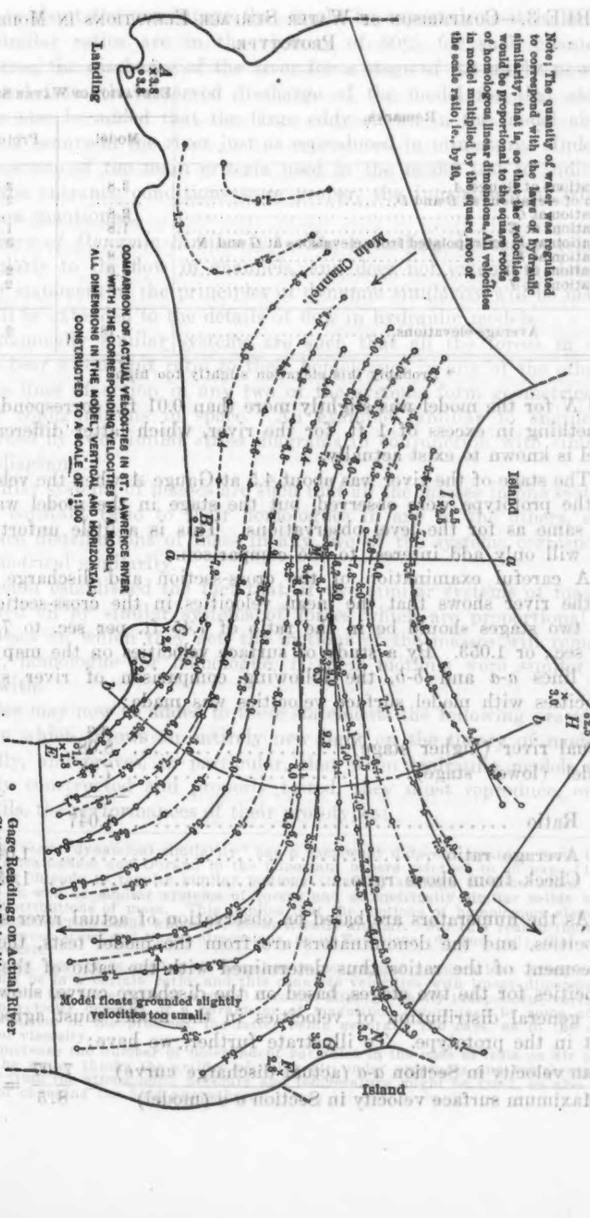

FIG. 10.

COMPARISON OF ACTUAL VELOCITIES IN ST. LAWRENCE RIVER WITH THE CORRESPONDING VELOCITIES IN A MODEL. ALL DIMENSIONS IN THE MODEL, VERTICAL AND HORIZONTAL, CONSTRUCTED TO A SCALE OF 1:100

Mr. Groat.

TABLE 3.—COMPARISON OF WATER SURFACE ELEVATIONS IN MODEL AND PROTOTYPE.

Remarks.	Elevation of Water Surface.	
	Model.	Prototype.
Elevations at Gauge *A*.	3.8	3.8
Mean of elevations at *B* and *D*.		2.65
Elevation at *C*.	2.8	....
Elevations at *E*.	1.8	1.5
Elevation at *F*, extrapolated from elevations at *G* and *N*.	....	3.1*
Elevation at *F*.	2.8	....
Elevations at *H*.	3.0	2.5
Elevations at *I*.	2.5	2.5
Average elevations.	2.78	2.68

* Probably this elevation slightly too high.

and N for the model was slightly more than 0.01 ft., corresponding to something in excess of 1 ft. for the river, which latter difference in level is known to exist actually.

The stage of the river was about 4.5 at Gauge A when the velocities in the prototype were observed, but the stage in the model was 3.8, the same as for the level observations. This is a little unfortunate, but will only add interest to the comparison.

A careful examination of the cross-section and discharge curve of the river shows that the mean velocities in the cross-section for the two stages should be in the ratio of 7.45 ft. per sec. to 7.07 ft. per sec., or 1.053. By a study of surface velocities on the map along the lines *a-a* and *b-b*, the following comparison of river surface velocities with model surface velocities was made:

	a-a	*b-b*
Actual river (higher stage)	8.08	7.99
Model (lower stage)	7.71	7.60
Ratio	1.047	1.051
Average ratio		1.049
Check from above ratio		1.053

As the numerators are based on observation of actual river surface velocities, and the denominators are from the model tests, the close agreement of the ratios thus determined with the ratio of the mean velocities for the two stages, based on the discharge curve, shows that the general distribution of velocities in the model must agree with that in the prototype. To illustrate further, we have:

$$\frac{\text{Mean velocity in Section } a\text{-}a \text{ (actual discharge curve)}}{\text{Maximum surface velocity in Section } a\text{-}a \text{ (model)}} = \frac{7.07}{8.5} = 83\%,$$

which is about the percentage that might be expected, as it is known that similar ratios are in the vicinity of 80% for such channels. Of course, the discharge of the river for a stage of 3.8 might now be computed from the observed discharge of the model for that stage. It may also be added that the large eddy shown in the model above Gauge *A* occurs in the river just as reproduced in miniature. Indeed, that was one of the main criteria used in the model test to indicate when the entrance conditions were proper, the importance of which has been mentioned. Mr. Groat.

*Theory of Dynamic Similarity.**—The preceding discussion relates particularly to the flow in channels, and does not extend to details. A brief statement of the principles of dynamic similarity will be made, and will be extended to the details of flow in hydraulic models.

Dynamically similar systems are such that all the forces in one system bear a constant ratio to their homologues in any of the others, and the lines of action in any two of the systems form geometrically similar configurations in space. An example familiar to engineers is afforded by two similar stress diagrams in conjunction with similar space diagrams.

Similar systems of masses are such that all the masses in one system bear a constant ratio to their homologues in any of the others, and the space distributions of mass in any two of the systems correspond to geometrical similarity.

Newton established the fact that, if two similar systems of masses are acted on by similar systems of forces which are proportional to the masses on which they act, the motions of the masses will remain similar, homologue for homologue, if their motions were similar to begin with.

There may now be added to these statements the following practical theorem which throws an entirely new light on the theory of models, generally, and proves, in particular, that when hydraulic models are properly constructed and properly tested, they must reproduce, even in details, the performances of their prototypes:

---

* The term "dynamical similarity" has a somewhat different meaning from that used by Buckingham and Durand in the important papers referred to on page 1172. The writer intends to require similar motions (that is, average motions which are similar) as well as similar systems of forces and geometrically similar solids with similar distributions of mass. This implies that the motions of homologous elementary masses describe paths which are geometrically similar. For this reason there is a bond between speed and linear dimensions, as in Equations (9) and (10), that cannot be broken.

Even if gravity is neglected, homologous centripetal accelerations (tangential also) must be in a certain ratio, and this connects velocities with linear dimensions. Consequently, the scale ratio of a model becomes either a wholly independent variable on which velocity depends or a parameter of magnitude determined by some additional condition of the problem, the latter being generally the case, as in the discussion of viscosity.

To increase the number of independent variables in the case of tests on air propellers the writer thinks the artifice of using air under pressures and temperatures differing from the atmospheric pressure and temperature might be tried, as also the artifice of changing the fluid altogether.

Mr. Groat.

When the velocities of homologous masses of floatage, water, suspended materials, and detritus, flowing similarly in two geometrically similar channels, are proportional to the square roots of homologous linear dimensions, then, according to kind, all the boundary and internal force reactions of practical moment, in both streams, form distinct pairs of similar systems of forces, any two homologues of which are in a fixed ratio, namely, that of corresponding homologous extraneous forces; provided the conditions of flow are sufficiently removed from those of pure stream-line flow to admit of the application of the ordinary theories of hydraulic friction, wind reactions, and transportation of detritus.

Therefore, when the theories apply, the similarity of motion must persist when once established.

When the proviso is satisfied, this theorem amounts to parallel integrations extended throughout the volume of the streams of masses and over their boundaries, including the free surfaces and terminal cross-sections. It follows that, if similarity of flow exists at the two terminal sections, the conditions of flow in the two channels will be similar, and the similarity will extend to details, the minuteness of which, in practical operations, will depend on the care and intelligence exercised in making use of the theory. It will be shown hereinafter that the proviso becomes redundant if the size of the model is properly determined.

To prove the theorem, the following notation will be adopted:

Notation.—Homologous quantities are represented by Italic or Greek capitals and their corresponding minuscules. Small letters relate to the model, capitals to the prototype. A Roman capital indicates the ratio of the two corresponding magnitudes, the numerators relating to the model, as $\mathrm{V} = v \div V$. The meanings of the characters are defined as follows:

a = area, homologue A, for example;

δ, Δ = densities of homologous masses;

f = force, primarily extraneous forces acting on a mass;

ϕ, Φ = force of skin friction;

g = acceleration of gravity at surface of earth;

γ, Γ = homologous accelerations when the masses are at different places, or when, by artificial means, the extraneous forces are made to differ from, or combine with, gravitational forces;

l = length;

m = mass;

μ, M = coefficients of viscosity for the case where different fluids flow in model and prototype;

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ν, N = kinematical coefficients of viscosity corresponding to above coefficients of viscosity; note that $\nu = \mu \div \delta$;

q = volume;

r = radius of curvature at point in path of moving mass;

s = specific weight of a stone, pebble, grain of sand; also a stress;

t = time;

ψ, Ψ = force of viscosity friction;

v = velocity;

w = weight;

ϖ, Ω = homologous specific weights.

Mass Ratio.—If two homologous masses in model and prototype are compared, supposing that solids of the same kind have a uniform density in the prototype and are represented by homologous masses of uniform density in the model, their ratio, of course, will be the same, whatever particular pair of homologues is selected. This is called the mass ratio, which is formally expressed by

$$\text{Mass ratio} = \mathbf{M} = \frac{m}{M} = \frac{\dfrac{\varpi}{\gamma} l^3}{\dfrac{\Omega}{\Gamma} L^3} = \frac{\delta\, l^3}{\Delta\, L^3} = \mathbf{D}\,\mathbf{L}^3 \ldots\ldots\ldots (6)$$

Force Ratio.—In similar manner, there is a fixed ratio for homologous extraneous forces, as for gravitational forces, and the theorem above shows that all other kinds of forces must appear in model and prototype in homologous pairs having this same ratio. Thus:

$$\text{(1) Force ratio} = \mathbf{F} = \frac{f}{F} = \frac{m\,\gamma}{M\,\Gamma} = \mathbf{M}\,\mathbf{G} = \mathbf{D}\,\mathbf{L}^3\,\mathbf{G} \ldots\ldots (7)$$

When $\mathbf{G} = 1$, it is plain that $\mathbf{M} = \mathbf{F}$, in Equation (6), which is the case when the tests are conducted at the same place on the earth's surface.

(2) *Centrifugal Reactions.*—All classes of forces must be related, so that the ratio of any two homologues will be equal to the force ratio, Equation (7). Thus, homologous centrifugal reactions will be in this ratio when the velocity ratio is determined by,

$$\mathbf{F} = \frac{m \dfrac{v^2}{r}}{M \dfrac{V^2}{R}} = \frac{\mathbf{M}\mathbf{V}^2}{\mathbf{R}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Mr. Groat. where r and R represent homologous radii of curvature for similar masses. As $r : R = l : L$, for similar paths we must have, from Equations (7) and (8):

$$\left.\begin{array}{r} \mathbf{M}\,\mathbf{G} = \dfrac{\mathbf{M}\,\mathbf{V}^2}{\mathbf{L}} \\ \text{or,} \qquad \mathbf{G} = \dfrac{\mathbf{V}^2}{\mathbf{L}} \end{array}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The interpretation of this last equation is that, when model and prototype are tested under different intensities of extraneous forces, homologous centripetal accelerations must be proportional to homologous gravitational accelerations, in order to preserve proper force relations.

If both model and prototype are tested under equal intensities of extraneous forces, as at the earth's surface, we have

$$\left.\begin{array}{r} \gamma = \Gamma = g \\ \text{or,} \qquad \dfrac{\gamma}{\Gamma} = \mathbf{G} = 1 \end{array}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

which proves that homologous centripetal accelerations are equal in such cases; that is $v^2 \div r = V^2 \div R$.

(3) *Proportionality of Heads and Depths.*—It is tacitly assumed in the theorem that homologous heads and depths are in the same proportion as homologous linear dimensions of the rigid portions of the model; that is, the volumes occupied by the water, or fluids, in model and prototype, are geometrically similar. It is necessary, therefore, to show that this assumption is not inconsistent with the requirements of hydraulic similarity; particularly that the hydraulic pressures and reactions on homologous portions of the boundary, or wet surface, meet the requirements of the force ratio, Equation (7). The ratio of any two homologous static pressures is

$$\frac{\omega\, h\, l^2}{\Omega\, H\, L^2} = \frac{\gamma}{\Gamma}\,\frac{\dfrac{\omega}{\gamma}\, l^3}{\dfrac{\Omega}{\Gamma}\, L^3} = \mathbf{G}\,\mathbf{D}\,\mathbf{L}^3 = \mathbf{F} \ldots\ldots\ldots\ldots (11)$$

by applying Equation (7).

Therefore, under the conditions imposed by Equation (9), the assumption of proportional homologous heads is consistent, whatever the size of the model.

(4) *Skin Frictions.*—If the surface of the model is geometrically similar to its prototype, the coefficients of skin friction for model and prototype are equal. Then the ordinary theory requires that the

resisting frictional forces along any two homologous surfaces are proportional to their areas and to the squares of the respective velocities along the surfaces, and therefore to the homologous areas and squares of any two homologous velocities. If the fluids in model and prototype are different, these frictions are altered in the direct ratio of densities. Therefore, the ratio of homologues is

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$$\frac{\phi}{\Phi} = \frac{\delta\, l^2\, v^2}{\Delta\, L^2\, V^2} = \mathrm{D\,L^3}\,\frac{\mathrm{V^2}}{\mathrm{L}} = \mathrm{D\,L^3\,G} = \mathrm{F} \text{............} (12)$$

by Equation (7).

Therefore, under the conditions imposed by Equation (9), homologous frictions satisfy the requirement of the force ratio, whatever the size of the model.

(5) and (7) *Wind Friction.*—All fluid friction follows laws substantially the same as for water. Then the same reasoning as used for skin friction can be applied to the case of homologous cakes of ice acted on by wind, and to the motion of the ice through the water by reason of that action. If the cakes of ice rise sufficiently above the surface of the water to offer obstruction to the wind, the resulting pressures are subject to laws similar to those yet to be discussed in connection with the pressures of water on stones and other detritus when lying on the river bed, where it is shown that such pressures satisfy the force ratio. Therefore, it may be accepted that wind reactions go with skin friction; one being shown to satisfy the requirement of hydraulic similarity, the other does also. Of course, it will be understood that wind velocities for the model are determined in the same manner as fluid velocities of any kind, that is, they are to be proportional to the square roots of homologous linear dimensions, when model and prototype are tested with one liquid at the same place. Just how to create a breeze for a model is a practical question which may be simplified by testing the model in the open air when the breeze is of the correct intensity. Thus, a 5-mile breeze would be equivalent to a 50-mile gale when the scale of the model is 1: 100.

(6) *Movement and Transportation of Detritus.*—This is one more action which must be heeded in constructing and testing a model of a channel which conducts water loaded with detritus. The density of any such materials is greater than that of water. Therefore, detritus diversion is the antithesis of ice diversion. Ice is floatage; detritus is "sinkage," to coin a new word (with apologies to Mr. Leighton).

The writer's experiments on models have included observations of the effects produced by currents in moving gravel and stones on the stream bed. To reproduce the effects in miniature, the river gravel must be sorted by screening so as to obtain the correct percentages of the different sizes. The gravel for the model is then screened and

Mr. Groat. mixed to correspond. The mean diameter of a model pebble, for example, must bear the same ratio to the mean diameter of the homologue in the actual stream that the linear dimensions of the model bear to the corresponding dimensions of the stream. If the model is not too small, imitation of performance under conditions of hydraulic similarity in the model will be surprising.

The average size of stones for a fill in swift water was determined by using a model, the experiments showing that the fill could be constructed across the stream up to a certain elevation which was the higher for the larger of any two sizes, and that the fill could be brought entirely above the water surface for sizes greater than a certain limiting size. Stones of the limiting size, and larger, now lie quiescent on the bed of the actual river at the place represented by the model. The model, too, was of a very small scale, only 1:100.

That such should be the facts may be proved theoretically. Merriman shows,* by simple assumptions, that the transporting capacity of a stream, measured by the weight of the heaviest stones which the current of water will move, is proportional to the sixth power of the impinging velocity. This proposition is accepted, with the qualification that it applies to geometrically similar shapes for stones of given material, of given density, and of consistent condition of surface. It is very evident that it could not apply, for example, to two stones one of which was spherical and the other cubical, or to two similar spalls of the same size and shape, one being clean and the other coated with slimy moss.

Merriman's assumptions imply that the force of a fluid impinging on an obstruction is jointly proportional to the obstructing area and the square of the impinging velocity. The writer extends the assumption by including the density of the fluid. When different fluids are flowing in model and prototype, and the latter are subject to different intensities of extraneous forces, the ratio of the transporting powers of the two streams, measured by the weight of the stones they will move, is

$$\frac{s}{S}\,\mathbf{L}^3 = \frac{\mathbf{S}}{\mathbf{G}}\,\mathbf{G}\,\mathbf{L}^3 = \mathbf{D}_s\,\mathbf{G}\,\mathbf{L}^3 \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

where $\mathbf{D}_s$ is the ratio of the densities of geometrically similar stones similarly situated in model and prototype. It is apparent, therefore, by Equation (7), that hydraulic and dynamic similarity can exist only when

$$\mathbf{D}_s = \mathbf{D},$$

that is, only when the densities of homologous stones are proportional to the densities of the fluids in model and prototype, homologue to

* "Treatise on Hydraulics" (1908), Art. 127, p. 323.

homologue. When the fluids are the same, $D = 1$, and homologous stones must be of the same density. The weights of the stones in Equation (13) form two similar systems of forces acting on homologous masses when $D_s = D$. Mr. Grost.

It still remains to be shown that the system of dislodgment forces also satisfies the force ratio, Equation (7), when the dynamic requirements of Equation (9) are imposed.

The assumptions require a dislodgment force ratio of

$$D\ A\ V^2 = D\ L^2\ V^2 = D\ L^3 \frac{V^2}{R} = D\ L^3\ G \qquad (15)$$

which shows that such forces are in harmony with hydraulic similarity by Equations (7) and (9).

Therefore, the ratio of the mean diameter of detritus transported by the model to that of detritus transported by the prototype is the same as the scale ratio, if Merriman's assumptions are correct. That is to say, if the model detritus is properly selected, sorted, proportioned, and introduced into the model under conditions of hydraulic and geometric similarity, the transporting phenomena of the stream should be reproduced in the model, a fact which is almost axiomatic, but, along with the theory of models, seems not to have had sufficient, if any, recognition.

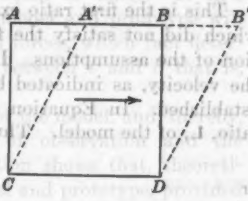

FIG. 11.

It should be carefully noted that homologous pebbles in model and prototype must be similarly situated on the beds of their respective channels.

Viscosity.—This property of fluids requires special attention in its relation to hydraulic models because its effects are not independent of the size of the model, as has been the case with all the other phenomena thus far discussed. Nevertheless, it is a subject which can be handled satisfactorily if theoretical relations are regarded.

As calculations involving viscosity are relatively infrequent, it will be well to refresh the memory as to these matters. Fig. 11 represents a small imaginary cube immersed in a liquid flowing in the direction of the arrow, the velocity of the fluid being graded uniformly in the increasing sense from the face, *CD*, to the face, *AB*. The value of the velocity along *CD* is v; along *AB* it is $v + dv$. If, then, the cube is imagined to move with the water so as to enclose constantly all the particles contained at the outset, a shearing distortion takes place, and the cube, or prism, becomes a parallelopiped, of cross-section *A′ B′ CD*.

Mr. Groat. If the face, AB, has the area, a, and the edge, $AC = BD$, is of length, dx, then experiment indicates that

$$\psi = \mu \, a \frac{dv}{dx},$$

where ψ is the viscosity friction, or force, on the area, a, necessary to produce the distortion, and μ is the coefficient of viscosity. Observe that this viscosity friction on the face, a, is proportional to the transverse velocity gradient, $dv \div dx$. Observe, also, that the dimensions of $dv \div dx$ are the same as those of $v \div l$. Therefore, the viscous reaction taking place between the contiguous masses of fluid subjected to shearing distortion has the dimensional form,

$$\psi = \mu \, a \frac{v}{l} = \mu \frac{l^2}{l} v = \mu \, l \, v \ldots\ldots\ldots\ldots (16)$$

Consequently, the force ratio for viscosity reactions is

$$\frac{\psi}{\Psi} = \frac{\mu \, l \, v}{M \, L \, V} = \mathbf{M'} \, \mathbf{L} \, \mathbf{V} \ldots\ldots\ldots\ldots (17)$$

where $\mathbf{M'} = \mu \div M$.

This is the first ratio examined, with the exception of Equation (8), which did not satisfy the force ratio, Equation (7), without modification of the assumptions. In Equation (8) it was necessary to regulate the velocity, as indicated by Equation (9), before similarity could be established. In Equation (17) it is necessary to determine the scale ratio, $\mathbf{L}$, of the model. Thus, equating Equations (7) and (17),

$$\mathbf{D} \, \mathbf{L}^3 \, \mathbf{G} = \mathbf{M'} \, \mathbf{L} \, \mathbf{V} \ldots\ldots\ldots\ldots (18)$$

or

$$\mathbf{L}^2 \, \mathbf{G} = \mathbf{N} \, \mathbf{V}$$

and

$$\mathbf{L}^{\frac{3}{2}} = \mathbf{N} \, \mathbf{G}^{-\frac{1}{2}} \ldots\ldots\ldots\ldots (19)$$

by applying Equation (9).

This shows that cases of pure stream-line flow cannot be reproduced in models unless the sizes of the models are properly determined by Equation (19).

When such models and their prototypes are tested under the influence of extraneous forces of equal intensity, $\mathbf{G} = 1$. When the fluid is the same in model and prototype, $\mathbf{N} = 1$. It follows that, in such cases, $\mathbf{L} = 1$, by Equation (19). Therefore, models will not reproduce similar conditions of flow when it is attempted to test them and their prototypes for stream-line configurations of water (any one fluid) at the same place on the earth's surface.

If, instead of eliminating $\mathbf{V}$, as in obtaining Equation (19), $\mathbf{G}$ is eliminated by Equation (9), there results,

$$\left.\begin{array}{l} \mathbf{L} \, \mathbf{V} = \mathbf{N} \\ \text{or,} \\ \dfrac{v \, l}{\nu} = \dfrac{V L}{N} \end{array}\right\} \ldots\ldots\ldots\ldots (20)$$

This appears to harmonize perfectly with Reynold's discovery for geometrically similar tubes, which states that $\frac{v_c d}{\nu}$ = constant, where v_c is Reynold's critical velocity and d is the diameter of the tube, ν being the kinematical coefficient of viscosity. It also appears to correspond very beautifully with Stanton and Pannell's plotting of the actual observations of the flow of water, air, and oil in pipes, the points all falling on one curve, irrespective of the fluid. Mr. Groat.

In the application of the theory to a model and prototype conveying the same fluid, it would appear, on superficial examination, that Equation (20) indicates similarity if the model is of any size, the velocity being regulated so as to be inversely proportional to the linear dimensions. This, however, is not the case, because the velocity is a function of the linear dimensions, by Equations (18) and (19), and not an independent variable in Equation (20), being, in reality, connected to the linear dimensions by Equation (9).

Thus we should not eliminate **G** from Equation (18), but **V**, or **L**, instead, because **G**, as a general thing, is equal to unity. If not, it should still be considered as fixed, as the model and prototype will be tested under fixed conditions of extraneous forces, which fact determines the value of **G**, and thus the relation between v and l, that is, the relation of **V** to **L**.

Equation (19)* determines the scale ratio of the model, and, thereby, the concomitant velocity when the places of observation and the particular fluids used are given. This equation shows that, theoretically, the same fluid may be used in both model and prototype, provided we are allowed to test under different intensities of extraneous forces. With **G** = 1, Equation (19) shows that the model requires a different fluid from that for the prototype when both observations are made at the same place.

In case of different fluids, with **G** = 1, **V** determined by Equation (9), and **L** determined by Equation (19), the extraneous forces, internal reactions, static and dynamic pressures, skin frictions, wind effects, resistances due to obstructions and transportation of detritus, transportivity and resistances due to floatage, and viscosity resistances form two dynamically and geometrically similar systems in which all pairs of homologous losses, including the total losses, are jointly proportional to densities, areas, and squares of homologous velocities. This does not mean that hydraulic gradients in the model or prototype are proportional to the squares of velocities, for they may be of any magnitude. For example, in pure stream-line flow the gradient is

* As examples of the uses of Equation (19), when **G** = 1 it may be shown: (*a*) that a model air propeller in mercury at 20° cent. should be one-twenty-fifth full size for the prototype in air at 15° cent.; (*b*) that the scale ratio of a model water-wheel tested in mercury at 20° cent., should be about 0.22 for the prototype in water at 15° cent. If the mercury can be at 100° cent., the scale ratio would be about one-seventh. The effect of changing the density of air for a model has not yet been investigated.

Mr. Groat. proportional to v. The theorem simply shows that whatever the slope in the prototype, the homologue in the model will be the same if the size of the model is properly chosen.

What we have done is to show that the conditions of geometric and dynamical similarity can be imposed simultaneously in a hydraulic model so that it will reproduce the prototype both in form and action, and that this possibility is in full accord with the generally accepted laws of turbulent stream flow, including viscosity resistances.

(8) and (9) *Observations and Measurements.*—If the duration of time observations on the prototype is always reduced in proportion to the square root of the scale ratio* (linear dimensions), then all velocities and quantities of discharge when measured to the reduced scale will be equal to the corresponding quantities measured to full-scale units in the prototype. The analogy may be made perfect by graduating a watch dial to time units reduced in the proportion of the square root of the scale ratio,* and providing measuring instruments, graduated to scale, for lengths, volumes, weights, etc. When this is done, all measurements taken on the model will be numerically equal to the homologues observed in the prototype with full-size measuring instruments and an ordinary timepiece. Such a procedure will eliminate many calculations, for example, calculating the discharge by the formula, $\mathbf{Q} = \mathbf{H}^{\frac{5}{2}}$. The actual time required for a sand-bar to form in a model built to a scale of 1:100, for example, will be only 2 days, when the corresponding bar in the river forms in 20 days. The time indicated on the specially graduated watch, however, will be 20 scale-days.

(10) *Factors of Safety in Models.*—In making models of structures to test to destruction, it is a mistake to use the same materials as in constructing the prototype. If the test is simply to determine relative stresses in the various members, it is unimportant what materials are used, if care is taken to see that elastic properties are correctly proportioned. If the model is for mere appearance, little or no heed need be given the internal qualities.

To examine the matter of stress in homologous members we use the method of dimensions, as heretofore. The dimensions of stress are,

$$s = \frac{f}{a} = \frac{\omega\, l^3}{l^2} = \omega\, l$$

or

$$\text{stress ratio} = \mathbf{S} = \frac{s}{S} = \frac{\omega}{\Omega} L$$

If the safe stresses of the materials of model and prototype are indicated by the subscript m, then the correct scale ratio of the model to develop equal factors of safety is determined by

$$\mathbf{L} = \frac{\Omega}{\omega} \frac{s_m}{S_m}$$

* When the intensities of extraneous forces differ between model and prototype the correct time ratio is $\mathbf{T} = \sqrt{\mathbf{L} \div \mathbf{G}}$.

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If, for any good reason, it is desired to build the model of the same materials as used in the prototype and still develop equal factors, this condition can be taken care of by increasing artificially the specific weight of the material of the model until the two specific weights are inversely proportional to the desired scale ratio, that is, $s_m = S_m$ and $L = \Omega \div \omega$.

This increase of the intensities of extraneous forces is most neatly accomplished by rotating the model as a whole about a vertical axis at considerable distance. The model must be placed, relative to the axis of rotation, so that the resultants of the action of gravity and the centrifugal reactions will be of the correct amounts and in the right direction relative to the model. Of course, the individual model structural members can be properly loaded to give the same results. The use of properly chosen materials for the model would seem to be the most scientific method.

The principal difficulty connected with the construction of a model with the same materials as used in prototype is that of securing model shapes of the same elastic properties as those of their homologues. A piano wire may have a strength of several hundred thousand pounds per square inch, whereas one would expect nothing of the sort in rounds of 2 or 3 in. diameter.

Professor Fletcher states that "the inexorable laws of mechanics forbid any such sweeping general conclusions." The writer's idea of mechanical laws is that they were devised expressly for the purpose of making such conclusions, and that they are themselves the most sweeping of all general conclusions.

Although model structures do not appear important as illuminating the question of hydraulic models, it is certain that model structures can be built and tested to show comparative results. It will be a service to negate to some extent any sweeping negative conclusion concerning their value.

It is perfectly proper to warn against all pitfalls connected with models. The real caution, however, is not against the use of models, but against their misuse.

Had the model water tank been constructed of appropriate materials, or, if necessary to test it when built of the materials used in constructing the prototype, had it been tested under proper conditions of loading, or of rotation, it would have developed the necessary stresses, and they would have been properly distributed among its members.

The elevated railway was not tested under conditions of mechanical similarity, as the speed of the engine was reduced by the direct scale ratio, instead of by its square root. If the radius of the full-size curve was 50 ft., as stated by Mr. Tratman, and the radius of the corresponding curve in the model 6¼ ft., as stated by Professor Fletcher, cor-

Mr. Groat. responding to a scale ratio of 1:8, then the proper speed ratio would be $1:\sqrt{8}$, or 1:2.83. Taking the velocity of 66 ft. per sec. for the full-size engine, as given by Professor Fletcher, the correct speed for the model should have been 66 ÷ 2.83 = 23.3 ft. per sec., approximately.

Had this speed been maintained at 23.3 ft. per sec. on a model structure properly built to scale, and on curves of any radius, homologous centrifugal reactions would be in proper proportion (the force ratio would be that of the weights of the two engines) and every member of the structure would receive its proper proportion of dead and live loads. Moreover, if the strengths of the materials were properly proportioned, the factors of safety in model and prototype would have been equal. It is not doubted that it would be something of an undertaking to construct and test such a model, but the fact which we have proved is that it can be done.

The writer is glad to note that Professor Fletcher finds the law of similarity satisfactory for the case of static pressures on a model dam. In the case of a dam and a geometrically similar model of one-ninth linear dimensions, the model operating under one-ninth the head on the prototype, the law of similar statics is proved by writing,

$$\text{static force ratio} = \frac{\omega}{\Omega} \times \mathsf{H}^2 \times \mathsf{H} = \frac{\omega}{\Omega}\,\mathsf{H}^3 = 9^3 \Big]_{\omega = \Omega}$$

$$\mathsf{H} = \frac{H}{h}$$

Then, for dynamic conditions, the corresponding relative discharges of the two similar dams would be $\mathsf{H}^{\frac{5}{2}}$, and the power ratio, evidently, $\mathsf{H}^{\frac{7}{2}}$. To obtain the relative dynamic forces acting on any two homologous areas of overfall, it is simply necessary to divide the power ratio by the velocity ratio, that is, by the subduplicate ratio of heads. Hence, the corresponding force ratio is,

$$\text{dynamic force ratio} = \mathsf{H}^{\frac{7}{2}} \div \mathsf{H}^{\frac{1}{2}} = \mathsf{H}^3 = 9^3,$$

which is just the same as that deduced for static conditions.

Therefore, homologous hydrodynamic forces and homologous hydrostatic forces are in the same ratio all over the entire wet areas of any two similar dams. Moreover, if the two similar dams are built of materials having the same density, then their weights, indeed their volumes, are also in this same ratio; namely, the force ratio of the homology. The dynamics of similar dams are consistent with the statics. It is entirely incorrect to refer the fall of water to the center of gravity of the overfall. This notion must be eradicated.

The writer thinks he has answered all objections in the discussion on detritus, and hopes Mr. Leighton will become sufficiently interested in models to have one actually tried out under conditions of hydraulic similarity.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1420

PROGRESS REPORT
OF THE SPECIAL COMMITTEE TO REPORT ON STRESSES IN RAILROAD TRACK*

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J. E. WILLOUGHBY

NOVEMBER 3D, 1917

* Presented to the Annual Meeting, January 16th, 1918

CONTENTS.

TO THE AMERICAN SOCIETY OF CIVIL ENGINEERS:

The Special Committee to Report on Stresses in Railroad Track herewith presents a progress report.

I.—INTRODUCTION.

1.—Preliminary.—The Committee was formed by action of the Board of Direction on November 12th, 1913. A similar committee, the Special Committee on Stresses in Track, was created by the Board of Direction of the American Railway Engineering Association on November 20th, 1913. In each case, authority was given to co-operate with the Committee of the other Society. The Committee of the American Railway Engineering Association consists of the members of the Special Committee of the American Society of Civil Engineers, sixteen in all, and five other members of the American Railway Engineering Association who do not hold membership in the American Society of Civil Engineers. The Committees of the American Society of Civil Engineers and of the American Railway Engineering Association, under the authorization given, have in effect carried on their work as a single committee. This report is presented simultaneously to the two Societies.

The Board of Direction of the American Society of Civil Engineers made an assignment of funds for the work of the Committee on March 4th, 1914, and a contribution toward the expenses of the tests was received by the American Railway Engineering Association a few days later. Funds having been provided to make a beginning, the initial meeting of the Committee was held in Chicago on March 18th, 1914, when the field of the work and the manner of undertaking it were discussed. At a meeting in Baltimore on June 3d, 1914, the report of a Sub-committee on Organization and Scope of Work was adopted. Mr. W. M. Dawley was made Vice-Chairman of the co-operating committees, and Mr. E. H. Fritch, Secretary of the American Railway Engineering Association, was elected Secretary. Preliminary field work was begun in the summer of 1914, and apparatus and methods of testing were developed as fast as experience could be gained. The work in field, laboratory, and office has since been carried on as actively as the conditions would warrant. Progress reports of the experimental work have been made to the Committee from time to time, and the results of the tests have been discussed at committee meetings and by correspondence. A great deal of time has been consumed in working up the results and in putting the data in form for presentation.

From the beginning the Committee has realized that the problem assigned to it is very complicated, involving many difficulties and uncertainties. It has felt that an adequate report on stresses in rail-

road track must be based on experimental data derived from extensive tests on standard railroad track. It has also realized that, because of the complexity of the action of track under load and the variability of the conditions which may be found in track and load, adequate experimental work would involve long, painstaking, and repeated tests under many conditions, and also that considerable time would be required for reducing the data thus obtained and interpreting the results. It believed that results of value could be obtained only after prolonged work on the problem. Experience has shown that the anticipated difficulties were not over-estimated. It was found necessary to expend considerable effort and time on the development of instruments for use in the tests and on methods for conducting tests. The problem was studied, and the methods were developed in the light of the information gained during the work. The experimental work undertaken thus far has included the measurement of track depressions, ballast and roadway pressure, and fiber stress in rail, for both static and moving loads, and laboratory tests on the distribution of pressure through ballast.

It is noteworthy that, notwithstanding the importance of track, relatively few experimental data on the general action of the track structure are on record and available as a guide to the Committee in its efforts, or for use in making comparisons with its results. The theoretical or analytical treatments which have been published, principally by European engineers, have most frequently been largely mathematical in character and without experimental verification or connection, and sometimes the assumptions on which the theory was based have been in error. Nevertheless, a number of engineers have made valuable tests on the action of track, notably Dr. P. H. Dudley in the United States, Cuenot in France, and Zimmerman and Ast in Germany and Austria, and these and others have contributed important discussions on several parts of the problem. It is planned to give a bibliography of the literature of the action of track as a structure under load in a later report.* It is apparent, then, that railroad track has not been developed in the manner followed in the development and expansion of most engineering structures and structural parts, where the scientific study of forces and stresses and the use of analysis and experiment have contributed in a marked way to improvements and growth. Instead, the present standards of track have been evolved from previous practice through a process involving extension and trial, and judgment and experience. That track has attained its present state of excellence is a tribute to the sense, insight, and judgment of the many men who have contributed to its growth and devel-

* A comprehensive list of the principal articles on this and related subjects up to 1909, presented by Dr. P. H. Dudley, may be found in *Proceedings*, Am. Ry. Eng. Assoc., Vol. 14 (1913), p. 570.

opment. It is not surprising, then, that the Committee found it necessary to devote considerable effort to studying the fundamental principles of the mechanics of track action, this being done principally through experimental work on track which may be described as ordinary track in good condition.

In the report, the action of track as an elastic structure is first taken up. The part on conduct of tests, which is next in order, includes a description of the apparatus and the method of conducting the tests, a statement of the preparation of test track and procedure of tests, and a discussion of reduction of data and accuracy of instruments. The results of tests are then presented, and various topics related to the action of track are discussed. The information obtained on the action of the tie and on the transmission of pressure through ballast and roadway will be given in a succeeding report.

The Committee is continuing work on the subject assigned to it.

2.—*Acknowledgment.*—The work of the Committee has been made possible through the contribution of funds by a number of corporations, which have in this way expressed appreciation of the importance of the work and the need for information. Acknowledgment is made to the United States Steel Corporation, the Bethlehem Steel Company, the Lackawanna Steel Company, and the Cambria Steel Company for their financial assistance. The American Society of Civil Engineers has also given support from its treasury.

Railroad companies have co-operated by furnishing facilities for the test work, and thus have contributed largely to the success of the undertaking. The Illinois Central Railroad, A. S. Baldwin, M. Am. Soc. C. E., Chief Engineer, gave the use of its tracks and of its locomotives and crews for the extensive set of tests made north of Champaign, Ill., and credit is due this company for participating so largely in the provision for the tests. The Delaware, Lackawanna and Western Railroad, George J. Ray, M. Am. Soc. C. E., Chief Engineer, gave the use of tracks, locomotives, and crews for the series of tests conducted near Dover, N. J. The Michigan Central Railroad provided the 100-lb. rails for the test track north of Champaign, and the Pennsylvania Railroad supplied the 125-lb. rails used.

Valuable assistance has been rendered by the University of Illinois. By a special arrangement, the Engineering Experiment Station has co-operated by giving the use of laboratory, shop, and office facilities, and through the service of its staff. Mr. H. F. Moore, Professor of Engineering Materials, has devoted considerable time and energy to the design and development of instruments used in the tests, and to the methods of testing. His special qualifications for this work have made his assistance most valuable. Credit should be given Dr. H. M. Westergaard, Instructor in Theoretical and Applied Mechanics, for his helpful work on the mathematical derivation of the equations

given in the analysis of the action of track on a continuous elastic support.

To Mr. H. R. Thomas, who, as Assistant Engineer of Tests, has been in immediate charge of the testing operations and of the office work, acknowledgment is due, not only for skill, industry, and insight, but for the zealous spirit of scientific investigation with which the work has been conducted. Mr. R. R. Zipprodt has contributed to the accuracy of the work by his faithful and caretaking reading of records, calculation of results, and assistance in field work. Necessarily, the personnel of the test party has been a changing one, but all those engaged in the work have exhibited care, loyalty, and enthusiasm, which has materially promoted the satisfactory progress of the investigation.

The committee was fortunate in having the advantage of the experience of Dr. P. H. Dudley, a pioneer in testing railroad track. Mr. P. M. LaBach's familiarity with European literature relating to track and track tests has been of service. Since its formation the Committee has lost by resignation of membership J. B. Berry and William McNab, Members, Am. Soc. C. E., whose duties in new positions prevented their giving further attention to the work of the Committee. A further loss in membership occurred through the death of Emil Gerber, M. Am. Soc. C. E.

II.—The Action of Track as an Elastic Structure.

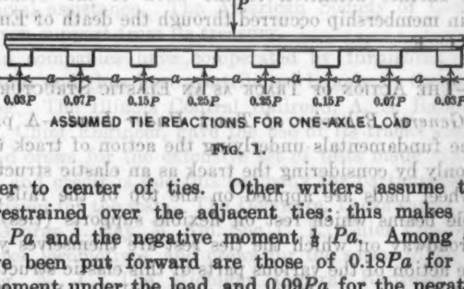
3.—The General Behavior of Track Under Load.—A proper conception of the fundamentals underlying the action of track under load may be had only by considering the track as an elastic structure under load: The wheel loads are applied on the top of the rails; the rails act as flexible beams which rest on flexible supports (ties); and the ballast and roadway on which the ties rest are themselves yielding or flexible. The action of the various parts of this elastic structure affects the action of other parts. It is evident that the quality of flexibility and elasticity and stiffness of the supporting substructure constitutes an important element in track action, influences very greatly the action of the rail, and affects the stresses developed in the various parts of the track.

Due to the stiffness of the rail and the yielding of its supports, the load from a wheel will be distributed over a number of ties. It is evident that the amount of yielding of the supports affects the values of the moments and stresses developed in the rail. As the distribution of the wheel load among a number of ties produces upward pressures or tie reactions of varying amounts, the determination of these tie reactions is connected with the problem of determining analytically the stresses in the rail. The properties of elasticity and stiffness in the rail, the tie, the ballast, and the roadway enter in a complex manner into the development of the stresses in

the track structure, the relative stiffness of the various parts affecting the results in any one part. The spacing of the wheels of locomotives and cars longitudinally along the track also influences the division of the load, as pressures on the various ties, and hence influences the value of the stresses developed in rails, ties, and ballast. The track is flexible, and its action under load is exceedingly complex, especially under the variable conditions found in tie bearings, ballast pressures, and roadway resistances.

Before attempting to give an analytical treatment of the moments and stresses developed in the parts of the track structure, it may be well to discuss in a general way the action of that structure. It is believed that this may be helpful in forming proper conceptions and in showing that certain notions of the manner in which the rail acts are incorrect and misleading.

A number of writers have obtained expressions for the bending moment and stresses in a rail by considering the latter as a simple beam supported on the two adjacent ties, with the wheel load at a point half way between. This assumption gives a positive bending moment under the wheel of $\frac{1}{4}$ Pa, where P is the wheel load and a is the distance



ASSUMED TIE REACTIONS FOR ONE-AXLE LOAD

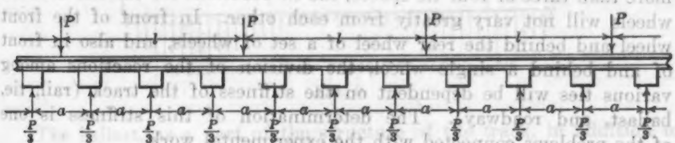
FIG. 1.

from center to center of ties. Other writers assume the beam to be fully restrained over the adjacent ties; this makes the positive moment $\frac{1}{8}$ Pa and the negative moment $\frac{1}{8}$ Pa. Among other values which have been put forward are those of $0.18Pa$ for the positive bending moment under the load, and $0.09Pa$ for the negative moment, which is considered to be over the adjoining tie. In all these it is virtually assumed that the load is taken only by the two ties adjacent to the load.

The conditions for a single wheel load may be approximated by taking the upward tie pressures as the loads on the rail and the wheel load as the reaction, and in effect considering the rail cut at the last tie. The wheel load is distributed over a number of ties. The proportional part of the wheel load for each tie reaction depends on the tie spacing, the stiffness of the tie and of the supporting substructure (ballast and roadway), and the stiffness of the rail. Consider that the tie reactions are somewhat as given in Fig. 1, which approximates the values found in tests. For this assumed distribution of the reactions, the bending moment under the load will then be $0.63Pa$, if we consider that the rail is not held down in such a way as to give

negative reactions and negative resisting moments away from the wheel. It is seen from this illustration that a single load may give a high bending moment in the rail.

If, now, we consider a rail with an indefinitely large number of evenly spaced wheel loads, the tie spacing being, say, not more than one-third of the wheel spacing, it can be shown that, for a given wheel spacing, there is relatively little difference in the tie reactions until the wheel spacing becomes quite large. Assuming for present purposes that the tie reactions are equal, and that the wheels are at points midway between ties (Fig. 2), the positive bending moment under loads three tie spaces apart is found to be about $0.263Pa$, and the negative bending moment $-0.154Pa$; and $0.345Pa$ and $-0.156Pa$, respectively, when the wheel spacing is four tie spaces. Expressing the bending moments in terms of the wheel spacing, l, the values become $0.088\ Pl$ and $-0.051\ Pl$ for $l = 3a$, and $0.086\ Pl$ and $-0.039\ Pl$ for $l = 4a$. It is readily seen from this that, for the conditions assumed, the bending moments—and therefore the stresses in the rail—will be dependent on the wheel spacing, and that this influence will continue until the wheel spacing becomes so great as to approach the conditions of single loads.



SERIES OF LOADS
WITH ASSUMPTION OF EQUAL TIE REACTIONS

Fig. 2.

The foregoing refers only to an indefinitely large number of wheel loads. For a small number the results will be greatly modified. In case two, three, or four wheels are spaced as above, it may be expected that the positive moment developed under the first and last loads will be greater than the foregoing values, and the moment under intermediate wheels somewhat less than at the outer wheels. The amount of this difference will depend on the properties of the track, including the stiffness of the rail. When there are other wheel loads, as truck wheels, not too far away, their effect is to modify the moments produced under the main wheels. On the whole, the problem is very complicated.

It is apparent that, for conditions here considered, changes in the closeness of tie spacing have relatively small effects on the bending moment developed in the rail by a given wheel loading; thus, for a wheel spacing equal to three tie spaces and an indefinitely large number of wheels thus spaced, the positive moment is about $0.088Pl$;

and, if the ties were spaced so closely as to approach conditions of uniform distribution of bearing pressures from wheel to wheel, the positive bending moment would be approximately $0.083Pl$. It is also well to note that, by this method of analysis, the bending moment under the wheel may be expected to be nearly as much for wheels placed over ties as when they are midway between ties. The foregoing discussion is based on an assumption of uniform resistance of the substructure; namely, that the stiffness of the elastic support is the same at each tie. It is evident that the stresses in the rail will be modified by the varying conditions of the tie, ballast, and roadway from point to point along the track.

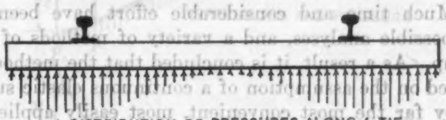
Another important element in the problem of determining stresses in track is the effect of speed of locomotive and train. On this matter analysis alone can give little real information, and it must be expected that reliance must be placed on experimental data.

The reaction of the tie or its upward pressure on the rail depends on the stiffness or the yieldability of the ballast and roadway, and also on the stiffness of the rail, which, of course, is dependent on its moment of inertia. For good track and for a wheel spacing of not more than three or four tie spaces, the tie reactions for the ties between wheels will not vary greatly from each other. In front of the front wheel and behind the rear wheel of a set of wheels, and also in front of and behind a single wheel, the division of the reactions among various ties will be dependent on the stiffness of the track (rail, tie, ballast, and roadway). The determination of this stiffness is one of the problems connected with the experimental work.

The distribution of the upward pressure of the ballast against a tie along its length may be expected to vary with the tamping and with the ballast and roadway conditions, and also depends on the dimensions of the tie. For usual conditions of good track, it would seem that the upward pressure will continue from the rail to the end of the tie and for at least a similar distance inside the rail. The exact distribution will depend on the amount of bending of the tie and on the permanent depression made in the ballast by the previous applications of load, as well as on the yieldability of the ballast and roadway. It will help in getting a conception of the action of the tie and ballast to consider the tie to be set on a bed of stiff springs; under load, the tie bends and the springs compress in proportion to the load put on them. The stiffness of the springs depends on the nature of the tie bearing and the consolidation of the ballast and other bearing material, and their elastic properties. At the middle point of the tie the springs will generally have little bearing resistance. Fig. 3 gives a general notion of the distribution of the load along a tie. However, it must be expected that changes in the conditions of ballast and roadway—like freezing and thawing—differences in

bearing conditions of adjacent ties, and the differences in resistance to repetition of pressure on the ballast along the length of the tie, greatly modify the distribution; and such changes may be expected to increase the bending stresses in the tie.

It is usual to think of the distribution of pressure across the width of the tie as being nearly uniform; that is, to consider the whole width of the tie as equally effective in transmitting pressure from the tie to the ballast. The ballast, however, is composed of non-cohesive particles. As the transmission of pressure in directions other than the vertical is dependent on friction between particles, the vertical pressures at the edge of the tie must be less than the pressures transmitted from the middle of its width, and this difference results in a distribution of pressure far from uniform across the width of the tie. For ties which are spaced not too far apart, there is something of a reaction from the adjoining tie, which may be expected to improve the distribution across the face. On the whole, however, the upward pressure against the tie cannot be uniform across its width.



A DISTRIBUTION OF PRESSURES ALONG A TIE

FIG. 3.

The ballast, as a part of the structure of the track, in addition to other important purposes, such as drainage, serves to transmit the tie pressures and reactions to the roadway, and acts to distribute the pressure more nearly evenly over the surface of the roadway. The laws governing the distribution and transmission of pressure through the ballast must be expected to have an important bearing on the proportioning of the track structure. The formulation of these laws and the determination of constants relating to stiffness or yieldability must await experimental work.

The roadway itself, acting as a support, also serves to transmit pressures and to distribute them to the surface of the earth below. The conditions of the roadway and the materials of which it is made may be expected to affect the action of the track. Little is known about this.

Enough has been stated to show that the problem of the stresses in track is not a simple one, that it involves a large number of elements which have the nature of variables, that these variables enter into the problem in a complex manner, and that it is necessary to have a diversity of experimental data relating to the several variables before attempting to formulate the laws governing the stresses in track.

4.—*Analysis of Track Action.*—An analytical treatment of the action of track under load has a value in the comparison of experimental data. It will be of use in establishing the physical properties of track. It should be an aid in forming proper conceptions of the manner in which track acts under load. It will be found to be a convenience in discussing the effect of rail section, tie spacing, driver and wheel spacing, depth of ballast and its stiffness, and condition of roadway. Such analyses are based on the theory of flexure, and involve complicated mathematical procedure. It may not be expected that an analysis of track action will fit accurately the many variables of track, and, besides, the track varies in its properties from point to point. What is wanted is an analysis that may readily be used and that may easily be applied to combinations of wheel loads, to variations in wheel spacing, and to particular physical conditions of track. Most analytical treatments give results in such complicated form that their application is time-consuming. The simplified forms of analysis may not have a wide range of applicability. Various methods of analysis have been examined, including those given in foreign publications. Much time and considerable effort have been devoted to a study of possible analyses, and a variety of methods of attack have been taken up. As a result, it is concluded that the method of analysis which is based on the assumption of a continuous elastic support under the rail is by far the most convenient, most easily applied, and most comprehensive in its application to the questions involved in the work of the Committee. This analysis lends itself very readily to a discussion of the general problems of track.

The assumption of a continuous support in place of tie supports is not an element of serious inaccuracy for the close tie spacing and large rail sections used on American railroads, especially as the use of values of track stiffness determined from data taken from tests on ordinary track carries into the constants established much of the condition of concentrated tie loads. The results of this analysis are similar in general character to those tried using concentrated tie loads. The method has been found to be more general and to have fewer limitations than the methods based on concentrated tie loads.

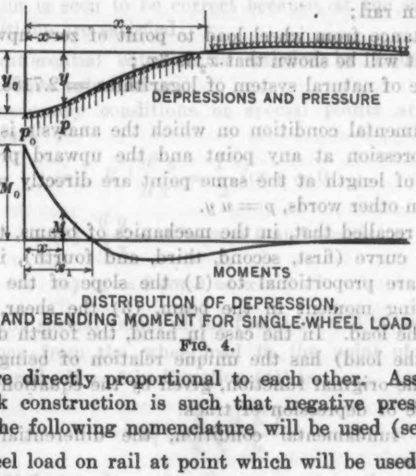
The term, modulus of elasticity of rail-support, is introduced as a measure of the vertical stiffness of the rail-support. It may be defined as the pressure per unit of length of each rail required to depress the track one unit. It represents the stiffness and yieldability of tie, ballast, and roadway, but does not involve the stiffness of the rail. As applied to ordinary track, the load on one rail required to depress one tie one unit, divided by the tie spacing, will give the modulus of elasticity of rail-support. As an illustration, consider that a series of wheel loads which depress the track an average of 0.3 in. gives a load equivalent to 10 000 lb. per tie for each rail, and that the tie

$E = 10000 \times \frac{1}{22} \times \frac{1}{0.3} = 1500$ lb in length of rail / per in of depression

spacing is 22 in.; the modulus of elasticity of rail-support is then 1 500 lb. per inch of length of rail per inch of depression. The value of the modulus of elasticity of rail-support is so related to tie spacing and tie dimensions, depth of ballast, quality of tie, solidity of roadway, character of tamping and surface, and other conditions of the track and its maintenance, that it will vary considerably according to the quality of the track.

The method of analysis will be developed, first for a single wheel load and then for a combination of wheel loads. (1)

5.—Depressions, Upward Pressures, and Bending Moments in Rail for Track having a Continuous Elastic Support; Single Wheel Load.—Assume that the rail is supported continuously on an elastic support and that the support has a constant modulus of stiffness; that is, that the depression of the track and the resulting upward pressures



DISTRIBUTION OF DEPRESSION, AND BENDING MOMENT FOR SINGLE-WHEEL LOAD

FIG. 4.

on the rail are directly proportional to each other. Assume, further, that the track construction is such that negative pressures may be developed. The following nomenclature will be used (see Fig. 4):

P = wheel load on rail at point which will be used as the origin of abscissas;

E = modulus of elasticity of steel;

I = moment of inertia of section of the rail;

y = depression of rail at any point, x, it being assumed that there is no play or back-lash in the track; downward displacement of rail is negative; however, in the applications to track, the ordinary downward depressions of track will be spoken of as positive;

y_0 = depression of rail at point of wheel load ($x = 0$);

p = upward pressure against rail per unit of length of rail at any given point;

p_0 = upward pressure against rail per unit of length of rail at the given wheel load ($x = 0$);

u = an elastic constant which denotes the pressure per unit of length of each rail necessary to depress the track (rail, tie, ballast, and roadway) one unit; for the system of units ordinarily used, it will be expressed in pounds per inch of length of rail required to depress the track 1 in.; u represents the stiffness of the track, and involves conditions of tie, ballast, and roadway; it is termed the modulus of elasticity of rail-support;

M = bending moment in rail at any point;

M_0 = bending moment in rail at point of wheel load due to single wheel load ($x = 0$);

x_1 = distance from wheel load to point of zero bending moment in rail;

x_2 = distance from wheel load to point of zero upward pressure; it will be shown that x_2 is $3x_1$;

e = base of natural system of logarithms = 2.7183.

The fundamental condition on which the analysis is based is that the track depression at any point and the upward pressure on the rail per unit of length at the same point are directly proportional to each other. In other words, $p = u\,y$.

It will be recalled that, in the mechanics of beams, the derivatives of the elastic curve (first, second, third, and fourth), in their order, represent or are proportional to (1) the slope of the elastic curve, (2) the bending moment in the beam, (3) the shear, and (4) the intensity of the load. In the case in hand, the fourth derivative (the intensity of the load) has the unique relation of being directly proportional to the original function, given by the equation of the elastic curve or curve of depression of track.

From the fundamental condition, the differential equation of equilibrium* is

$$E\,I\,\frac{d^4 y}{d\,x^4} = u\,y \qquad (1)$$

This differential equation is satisfied by the following equation:

$$y = -\frac{P}{\sqrt[4]{64\,E\,I\,u^3}}\,e^{-x\sqrt[4]{\frac{u}{4EI}}}\left(\cos.\,x\sqrt[4]{\frac{u}{4EI}} + \sin.\,x\sqrt[4]{\frac{u}{4EI}}\right) \qquad (2)$$

* The method of attack used in this mathematical solution may be found in Föppl's "Technische Mechanik", Vol. III, pp. 254–268, Edition of 1900, where a treatment of various problems concerning beams on continuous elastic supports is given. The method of analysis is here applied directly to the problem of track, and especially to the bending moment in the rail.

The successive derivatives of this expression are:

$$\frac{dy}{dx} = \frac{2P}{\sqrt[4]{16\,E\,I\,u}}\, e^{-x\sqrt[4]{\frac{u}{4EI}}}\ \sin.\, x\sqrt[4]{\frac{u}{4\,E\,I}} \quad \ldots\ldots\ldots (3)$$

$$M = E\,I\frac{d^2\,y}{d\,x^2} = P\sqrt[4]{\frac{E\,I}{64\,u}}\, e^{-x\sqrt[4]{\frac{u}{4EI}}}\left(\cos.\, x\sqrt[4]{\frac{u}{4\,E\,I}} - \sin.\, x\sqrt[4]{\frac{u}{4\,E\,I}}\right) \quad (4)$$

$$E\,I\frac{d^3\,y}{d\,x^3} = -\frac{P}{2}\, e^{-x\sqrt[4]{\frac{u}{4EI}}}\ \cos.\, x\sqrt[4]{\frac{u}{4\,E\,I}} \quad \ldots\ldots\ldots\ldots (5)$$

The equation for intensity of pressure against the rail is

$$p = -u\,y = P\sqrt[4]{\frac{u}{64\,E\,I}}\, e^{-x\sqrt[4]{\frac{u}{4EI}}}\left(\cos.\, x\sqrt[4]{\frac{u}{4\,E\,I}} + \sin.\, x\sqrt[4]{\frac{u}{4\,E\,I}}\right) \quad (6)$$

The mathematical derivation of these equations will not be given, but the solution is seen to be correct because, at the same time, two types of conditions are satisfied:

1.—The differential equation is satisfied by Equation (2) for all positive values of x.

2.—All necessary conditions at special points are satisfied as follows:

(a) $E\,I\dfrac{d^3\,y}{d\,x^3} = -\dfrac{P}{2}$ for $x = 0$

(b) $\dfrac{d\,y}{d\,x} = 0$ for $x = 0$ and $x = \infty$

(c) $y = 0$ for $x = \infty$

The following special values of the functions will be useful:

The distance from the wheel load to the point of zero bending moment in the rail ($M = 0$ in Equation (4)) is

$$x_1 = \frac{\pi}{4}\sqrt[4]{\frac{4\,E\,I}{u}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The value of the maximum bending moment in the rail (which is at the wheel load) ($x = 0$ in Equation (4)) is

$$M_0 = P\sqrt[4]{\frac{E\,I}{64\ u}} = 0.318\,P\,x_1 \quad \ldots\ldots\ldots\ldots\ldots (8)$$

The value of the maximum track depression (which is at the wheel load) ($x = 0$ in Equation (2)) is

$$y_0 = \frac{P}{\sqrt[4]{64\,E\,I\,u^3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The value of the maximum intensity of upward pressure (which is at the wheel load) ($x = 0$ in Equation (6)) is

$$p_0 = P \sqrt[4]{\frac{u}{64\,E\,I}} = -\,u\,y_0 \qquad (10)$$

The distance from the wheel load to the point of zero upward pressure on the rail ($p = 0$ in Equation (6)) is

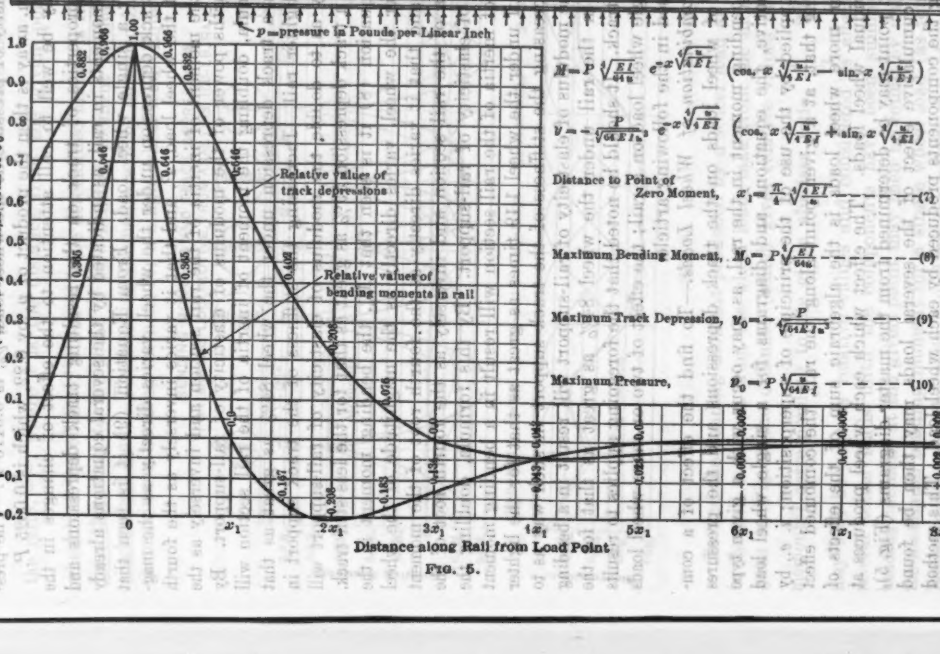
$$x_2 = \frac{3\,\pi}{4} \sqrt[4]{\frac{4\,E\,I}{u}} = 3\,x_1 \qquad (11)$$

Fig. 5 gives master diagrams for (1) the moment developed in the rail, and (2) the intensity of pressure against the rail and the depression of the rail, as determined from the foregoing equations. The diagrams represent relative values. The bending moment under the wheel load is given as unity, and the values of the bending moment at other points (given as ordinates of the bending moment curve) are expressed in terms of the bending moment at the wheel load. The intensity of upward pressure (represented by the ordinate to the pressure curve) is also expressed in terms of the intensity of pressure at the wheel load as unity. Similarly, the abscissas of the two diagrams (bending moments and pressures) are expressed in terms of the distance from the wheel load to the point of zero bending moment as unity; *i. e.*, as abscissa ratios. For any given track conditions, the value of the maximum bending moment, M_0, may be computed by Equation (8) and the distance, x_1, to the point of zero bending moment by Equation (7). To get the bending moment at any given point, find the ratio of x for the given point to x_1 and take the ordinate from the diagram by using this ratio as the abscissa. The bending moment in the rail at the given point will then be found by multiplying the value of the maximum moment, M_0 (already computed), by the ordinate found from the diagram. Similarly, the value of the intensity of pressure at a given point may be determined by multiplying the intensity of pressure under the wheel load by the ordinate for pressure at a given point obtained from the diagram.

As an illustration, consider a 100-lb. rail section ($I = 44$) and that, for the track conditions, $u = 1\,500$ lb. per inch of length of rail per inch of depression. x_1 is found to be 34 in. For a point 66 in. from the wheel load, the abscissa ratio is then $\frac{x}{x_1} = \frac{66}{34} = 1.94$. From the master diagrams (Fig. 5), the bending-moment ratio at the abscissa ratio, 1.94, is $-$ 0.20; that is, the bending moment at 66 in. from the wheel load is 0.20 of that at the wheel load, and is negative. By Equation (8) the maximum moment, M_0, is $10.8P$. The bending moment at a point 66 in. from the

MASTER DIAGRAM

FOR (1) MOMENT DEVELOPED IN THE RAIL, AND (2) INTENSITY OF PRESSURE AGAINST THE RAIL, AND DEPRESSION OF THE RAIL, SINGLE-WHEEL LOAD.



$$M = P\sqrt[4]{\frac{EI}{64u}}\, e^{-x\sqrt[4]{\frac{u}{4EI}}}\left(\cos.\, x\sqrt[4]{\frac{u}{4EI}} - \sin.\, x\sqrt[4]{\frac{u}{4EI}}\right)$$

$$v = -\frac{P}{\sqrt[4]{64EIu^3}}\, e^{-x\sqrt[4]{\frac{u}{4EI}}}\left(\cos.\, x\sqrt[4]{\frac{u}{4EI}} + \sin.\, x\sqrt[4]{\frac{u}{4EI}}\right)$$

Distance to Point of Zero Moment, $x_1 = \frac{\pi}{4}\sqrt[4]{\frac{4EI}{u}}$ ---------- (7)

Maximum Bending Moment, $M_0 = P\sqrt[4]{\frac{EI}{64u}}$ ---------- (8)

Maximum Track Depression, $v_0 = -\frac{P}{\sqrt[4]{64EIu^3}}$ ---------- (9)

Maximum Pressure, $p_0 = P\sqrt[4]{\frac{u}{64EI}}$ ---------- (10)

FIG. 5.

wheel load is then the product of M_0 and — 0.20, which is — 2.16P. For the same point ($x = 66$ in.), the relative value of the intensity of pressure is found from the diagram to be 0.22. By Equation (10) the intensity of pressure at the wheel load, p_0, is 0.0115 P. The pressure 66 in. away is then the product of p_0 by 0.22, which is 0.0025 P.

It may be well to call attention to the effect of changes in the various properties of track on the resulting track depressions and bending moments in rail, as indicated by the several equations already given for a single wheel load. From Equation (9) it is seen that y_0, the track depression under the wheel, varies directly as the magnitude of the wheel load, P, and that it varies inversely as the fourth root of the moment of inertia of the rail section and inversely as the three-fourths power of the modulus of elasticity of rail-support. By this formula, doubling the moment of inertia of the rail section will result in a track depression under the wheel 84% as great as that for the lighter rail. Increasing the stiffness of the track support in such way as to double the modulus of elasticity of rail-support will result in a track depression 59% as great as that for the less stiff track. From Equation (8) it is seen that M_0, the bending moment in the rail under the wheel, varies directly as the magnitude of the wheel load, P, and that it varies directly as the fourth root of the moment of inertia of the rail section and inversely as the fourth root of the modulus of elasticity of rail-support. By this formula, doubling the moment of inertia of the rail section will result in a bending moment in the rail under the wheel 1.19 times as great as that for the lighter rail. Increasing the stiffness of the track support in such way as to double the modulus of elasticity of rail-support will result in a bending moment in the rail under the wheel 84% as great as that for the less stiff track. It should be noted that the foregoing applies to results for a single wheel load on a rail; the effect of two or more wheel loads is discussed in the following article.

6.—Combination of Wheel Loads.—To find the effect of a combination of wheel loads on the track depressions and the pressures and the bending moment in the rail, as may occur with a given type of locomotive, the equations and diagrams for a single wheel load may be applied by the use of the principle of superposition; *i. e.*, by considering that, at a given point along the rail, the combined effect of two or more wheel loads is the algebraic sum of the effects of the individual wheel loads. The effect which each wheel produces at any given point may be determined from the master diagrams (Fig. 5), and the cumulative effect of the several loads may then be found by adding the components produced by each wheel load. This method of superposition of components is applicable to bending moments as well as to track depressions and pressures. The method may best be described by an illustrative example.

Take a Mikado locomotive, with the wheel loads and wheel spacings given in Fig. 6, and let the problem be to calculate the bending moments in the rail at various points. In using Fig. 5, take as the abscissa the ratio, $\frac{x}{x_1}$, where x is the distance from the given point to the wheel load the effect of which is to be determined and x_1 is the distance from the wheel to the point of zero bending moment in the case of a single wheel load. The bending moments will be obtained in terms of the maximum bending moment for a single wheel load (Equation (8)).

BENDING MOMENTS AND TRACK DEPRESSIONS
FOR MIKADO LOCOMOTIVE, FROM ANALYSIS. 100-LB. RAIL

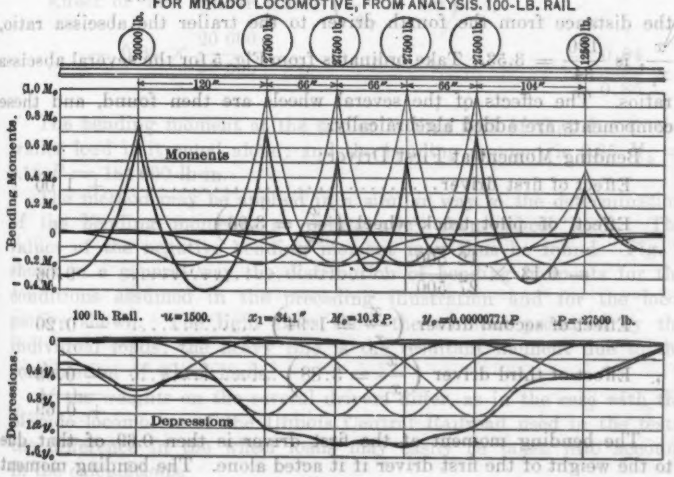


FIG. 6.

To find the bending moment produced by a wheel load at a section in the rail x distant from the wheel load, take from the moment diagram in Fig. 5 the ordinate (which gives relative values of bending moments) corresponding to the given abscissa ratio, $\frac{x}{x_1}$. The values may best be kept as relative values and considered as effects of the several wheel loads until the components have been summed. The bending moment at the given point may then be found by multiplying the maximum bending moment for a single wheel load (as found from Equation (8)) by the ratio found by the summation of components. Where the wheel loads are unequal, use the ratio of the wheel loads in the calculations; thus, if the effect of the pilot truck wheel on a point at the first driver is to be found, use the ratio $\frac{12\ 500}{27\ 500}$.

Consider that the track conditions are such that the modulus of elasticity of rail-support, u, is 1 500 lb. per inch of length of rail per inch of depression. Consider a 100-lb. rail section, with $I = 44$. The value of the bending moment under a wheel load, where uninfluenced by other wheel loads, becomes, for the constants of the track, from Equation (8), $M_0 = 10.8\ P$. From Equation (7), $x_1 = 34$ in. For the distance from the first driver to the pilot truck wheel the abscissa ratio, $\frac{x}{x_1}$, is $\frac{104}{34} = 3.06$. For the distance from the first driver to the second driver the abscissa ratio, $\frac{x}{x_1}$, is $\frac{66}{34} = 1.94$. For the distance from the fourth driver to the trailer the abscissa ratio, $\frac{x}{x_1}$, is $\frac{120}{34} = 3.53$. Take ordinates from Fig. 5 for the several abscissa ratios. The effects of the several wheels are then found, and these components are added algebraically.

Bending Moment at First Driver:

Effect of first driver	$+ 1.00$
Effect of pilot truck wheel $\left(\frac{x}{x_1} = 3.06\right)$	
$- 0.13 \times \frac{12\ 500}{27\ 500}$	$- 0.06$
Effect of second driver $\left(\frac{x}{x_1} = 1.94\right)$	$- 0.20$
Effect of third driver $\left(\frac{x}{x_1} = 3.88\right)$	$- 0.05$
	$+ 0.69$

The bending moment at the first driver is then 0.69 of that due to the weight of the first driver if it acted alone. The bending moment at this point becomes $M = 0.69\ M_0 = 7.45\ P = 205\ 000$ lb-in.

Bending Moment at Second Driver:

Effect of second driver	$+ 1.00$
Effect of first driver $\left(\frac{x}{x_1} = 1.94\right)$	$- 0.20$
Effect of third driver $\left(\frac{x}{x_1} = 1.94\right)$	$- 0.20$
Effect of fourth driver $\left(\frac{x}{x_1} = 3.88\right)$	$- 0.05$
Effect of pilot truck wheel $\left(\frac{x}{x_1} = 5.0\right)$	
$- 0.0 \times \frac{12\ 500}{27\ 500}$	0.00
	$+ 0.55$

The bending moment at the second driver is then 0.55 of that due to the weight of the second driver if it acted alone. The moment at this point becomes $M = 0.55\ M_0 = 5.94\ P = 163\,000$ lb-in.

Bending Moment at Trailer:

Effect of trailer load.............................. $+\ 1.00$

Effect of fourth driver $\left(\frac{x}{x_1} = 3.53\right)$

$-\ 0.08 \times \frac{27\,500}{20\,000}$ .... $-\ 0.11$

Effect of first tender wheel $\left(\frac{x}{x_1} = 4.0\right)$

$-\ 0.04 \times \frac{20\,000}{20\,000}$ .... $-\ 0.04$

$+\ 0.85$

The bending moment at the trailer is then 0.85 of that due to the trailer load if it acted alone; and the bending moment is $0.85\ M_0 = 9.18\ P = 184\,000$ lb-in.

The method may be applied in a similar way to the determination of the bending moments at points away from wheel loads. The values of the negative bending moment may thus be found. Fig. 6 shows in a general way the distribution of bending moments for the conditions assumed in the preceding illustration and for the locomotive shown. The light lines show the moment produced by the individual loads; the heavy line is the resultant moment due to the combination of wheel loads.

If the weights on the several drivers differ, as in the case with the Mikado locomotive of the Illinois Central Railroad used in the tests, the difference in the wheel loads may easily be taken into account in the calculations.

The track depressions caused by the assumed locomotive loading may be calculated by a similar process. At any point along the rail the components of track depressions caused by the individual wheel loads are found, and their algebraic sum gives the track depression resulting from the combination of loads. The values may be taken from Fig. 5 (which gives track depressions in terms of y_0, the depression under a single wheel load) for any given value of $\frac{x}{x_1}$, where x is the distance from the given point on the rail to the wheel load the effect of which on the track depression is to be determined, and x_1 is the distance from the wheel to the point of zero bending moment, in the case of a single wheel load. The values of the ratios thus found for the effect of the several loads may then be added algebraically; the product of this resultant ratio and y_0 is the resultant track depres-

sion due to the combination of loads. Fig. 6 illustrates the track depression for the Mikado locomotive used in the example illustrating the method of calculating bending moments. The light lines show the track depression produced by the individual wheel loads; the heavy line is the resultant track depression due to the combination of wheel loads. To illustrate the method of calculation, consider the case of the track depression under the first driver, as follows:

Effect of first driver .... $+ 1.00$

Effect of pilot truck wheel $\left(\frac{x}{x_1} = 3.06\right)$

$$-0.01 \times \frac{12\,500}{27\,500} \ldots\ldots\ldots\ldots \quad -0.00$$

Effect of second driver $\left(\frac{x}{x_1} = 1.94\right)$ .... $+ 0.23$

Effect of third driver $\left(\frac{x}{x_1} = 3.88\right)$ .... $- 0.04$

$+ 1.19$

The track depression at the first driver is then 1.19 times that due to the weight of the first driver if it acted alone. Since y_0, the depression under the first driver acting alone (from Equation (9)), would be $0.00000774\ P = 0.213$ in. for the conditions already assumed for the track, the track depression at this point will be $1.19\ y_0 = 0.254$ in. The heavy line in Fig. 6 gives the resulting track depressions at all points along the locomotive. The vertical pressures may be found from the track depressions by the use of the modulus of elasticity of rail-support, u.

Figs. 7 and 8 give values of bending moment in rail and track depression for an Atlantic locomotive having the wheel loads and wheel spacing shown, the former for the worn 85-lb. rail section used on the test track, and the latter for the 125-lb. rail section, the value of u being 1 500 lb. per in. per in.

Fig. 9 gives values of bending moment in rail and track depression for a single car having the loads shown, and Fig. 10 gives values for the truck wheels of two adjacent cars, the rail being the worn 85-lb. section of the test track on the Illinois Central Railroad and the value of u being 1 500 lb. per in. per in.

It is seen that, by the method here described, the effect of a combination of two or more wheel loads on the track depression and the bending moment in rail may be determined for the condition of continuous support here assumed. It is evident that a second wheel load placed within a limited distance from another wheel will cause a smaller bending moment to be produced in the rail at the wheel, and likewise a larger track depression than would be produced with

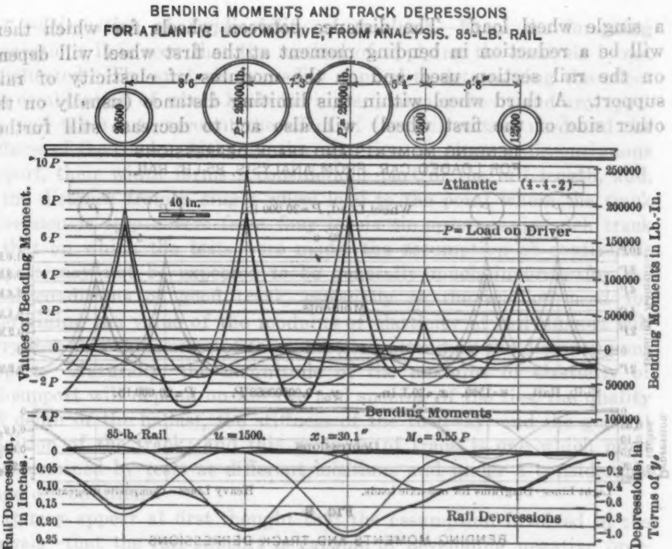

BENDING MOMENTS AND TRACK DEPRESSIONS
FOR ATLANTIC LOCOMOTIVE, FROM ANALYSIS. 85-LB. RAIL
Atlantic (4-4-2)
P= Load on Driver
40 in.
Values of Bending Moment.
Bending Moments in Lb.-In.
Bending Moments
85-lb. Rail
u =1500.
x1 =30.1"
M0 = 9.55 P
Rail Depression, in Inches.
Depressions, in Terms of y0
Rail Depressions

FIG. 7.

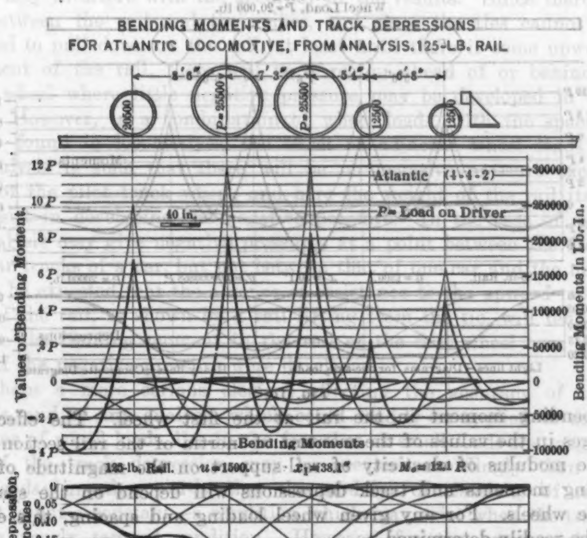

BENDING MOMENTS AND TRACK DEPRESSIONS
FOR ATLANTIC LOCOMOTIVE, FROM ANALYSIS. 125-LB. RAIL
Atlantic (4-4-2)
P= Load on Driver
40 in.
Values of Bending Moment.
Bending Moments in Lb.-In.
Bending Moments
125-lb. Rail.
u = 1500.
x1 =38.1"
M0 = 12.1 P.
Rail Depression, in Inches
Rail Depressions

FIG. 8.

a single wheel load. The distance between wheels for which there will be a reduction in bending moment at the first wheel will depend on the rail section used and on the modulus of elasticity of rail-support. A third wheel within this limiting distance (usually on the other side of the first wheel) will also act to decrease still further

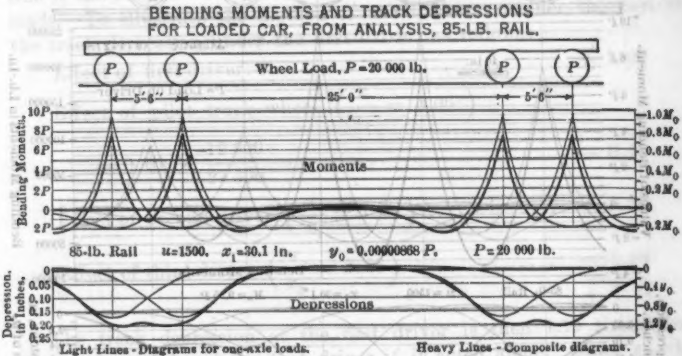


Fig. 9.

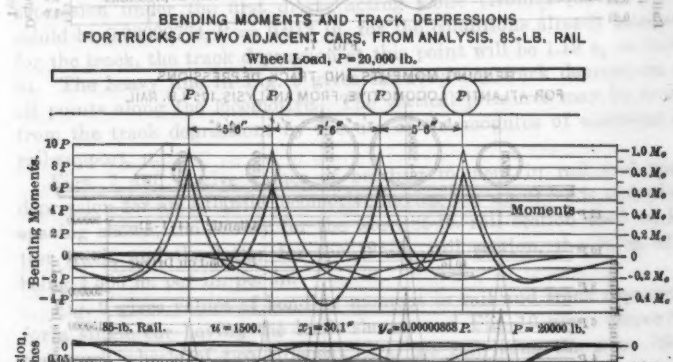


Fig. 10.

the bending moment in the rail at the first wheel. The effect of changes in the values of the moment of inertia of the rail section and of the modulus of elasticity of rail-support on the magnitude of the bending moments and track depressions will depend on the spacing of the wheels. For any given wheel loading and spacing, this effect may be readily determined.

7.—*Discussion of Applicability of Analysis.*—From the foregoing illustrations it is seen that the method of applying the analysis is not complicated. The analysis is readily applied to determining the effect of a combination of wheel loads having any arrangement and any spacing, and takes into account the effect of the rail section used and the stiffness of the track. Although the cross-ties constitute a discontinuous support, their width forms a considerable part of the rail length, and, as the distance from a single wheel load to the point where the track depression is zero covers from four to six tie spacings, in such track as that on which the tests were made, the assumption of continuous support may not be expected to be generally much in error for the usual conditions of good track, especially as the method used for determining the value of the modulus of elasticity of rail-support will carry into this value some of the conditions attending the discontinuous support. Naturally, the magnitude of the modulus of elasticity of rail-support will depend on the size and spacing of the ties, the quality and depth of the ballast, the stiffness of the roadway, and the general condition of the track; and this property of track is one which must be determined by tests at different locations and under a considerable range of conditions.

It may appear at first thought that the assumption involved in the analysis—that the rail support is capable of developing negative pressures—may interfere with the accuracy of the results. Since there is play between the rail and the spikes, and since the ties cannot be expected to pull down on the rail, at least until there is some upward movement of the rail, there will be a region ahead of or behind a single wheel where little negative pressure may be developed in the track. However, in a combination of wheel loads, with the spacing usually found in locomotives, the effect of adjacent wheel loads in the analysis is such that there will be no negative pressure except ahead of the pilot truck wheel, and here the weight of the rail itself will assist in developing the negative pressure. In the case of cars, the analysis may give negative pressures at a point between the front and rear trucks of a car, but not between that of one car and the next. It may be expected that the absence of resistance to the upward movement of the rail, as shown by a rail lifting from the tie, will tend to increase the bending moment in the rail at the first wheel over that given if the negative pressure were developed.

Perhaps a more serious element affecting the accuracy of this analysis, and of any analysis which could readily be applied to combinations of wheel loads, lies in the fact that at small loads the magnitude of the track depression may be greater accordingly than at large loads; in other words, that the modulus of elasticity of rail-support is not a constant—a condition which may exist when the track is in a poorly tamped condition. However, for the heavier wheel

loads, like the drivers and trailer, this influence may not be very important. For a condition where the modulus of elasticity of rail-support is not a constant, the principle of superposition, which is utilized in most methods of analysis involving flexure of structures, is not applicable, and hence any treatment based on the actual relation between pressure and depression will be altogether too complicated for any practical use.

It may not be expected that any method of analysis will give results that will fit accurately with experimental values. The variables of track are numerous and uncertain. Notwithstanding this, a usable analytical treatment is of great value in the comparison of experimental data and in acquiring a conception of the fundamentals of track action. It is believed that the analysis here given is acceptable from most points of view. Possibly in the future it may be found practicable to make empirical modifications of it which will fit more closely the data of tests.

III.—The Conduct of Tests.

A.—Apparatus, and Method of Conducting Tests.

8.—General Requirements for Test Apparatus.—An experimental study of stresses in railroad track involves the measurement of deformations or strains in rails, pressures existing at various points throughout the ballast, deflection of rails, depression and bending of ties, and depression at various points in the ballast and the roadway, when acted on by a load. As the study of the effects of different kinds of loading, such as one-axle loading, two-axle loading, loading with locomotives of different wheel spacing applied both statically and at various speeds, was an essential part of the undertaking, it was necessary that the design of the instruments be made suitable for use under the diverse conditions of the various tests.

In the design of instruments, simplicity and reliability in operation rather than extreme sensitiveness were considered the prime requisites for measurements of strains, pressure, and depression. The instruments must be designed so that they will operate satisfactorily under variable weather conditions and give reliable results when used amid considerable dust and dirt. Some instruments are attached to rails subjected to severe vibration under rapidly moving locomotives; others are of necessity left buried in ballast for weeks at a time. As thousands of observations were taken, and as the instruments had to be adjusted or moved frequently in the short time between the passage of trains on a busy track, the importance of simplicity and rapidity of operation can hardly be over-emphasized. The dust and dirt unavoidably present would greatly lessen the reliability of instruments using complicated, delicate mechanism.

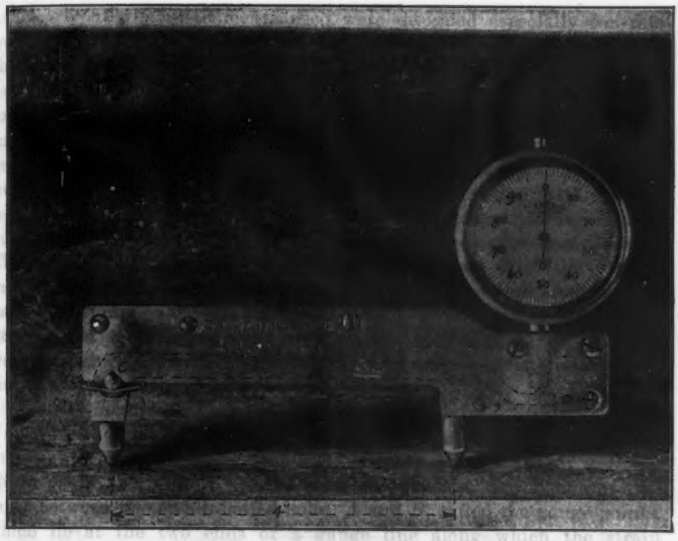


FIG. 11.—BERRY STRAIN GAUGE.

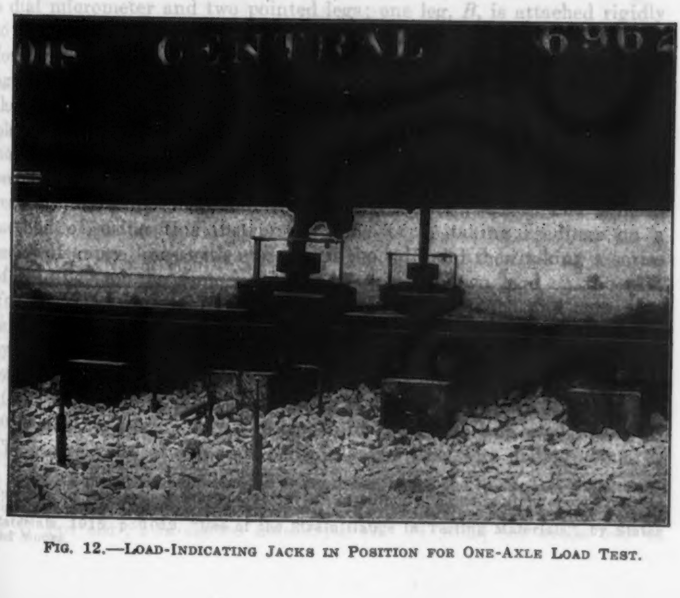


FIG. 12.—LOAD-INDICATING JACKS IN POSITION FOR ONE-AXLE LOAD TEST.

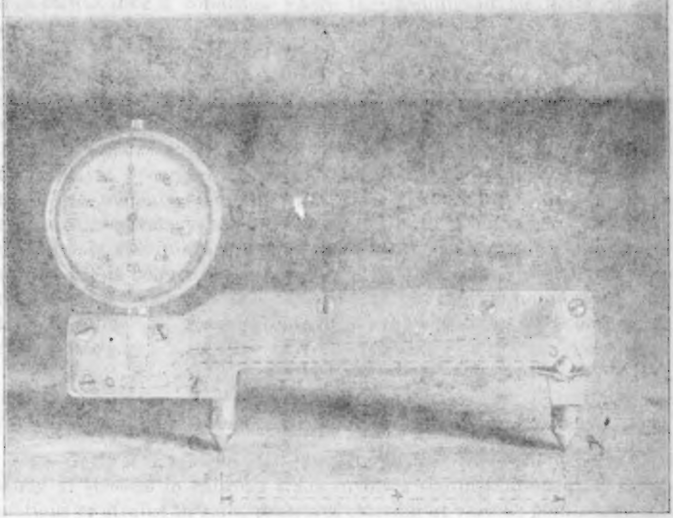


Fig. 11.—Berry Strain Gauge.

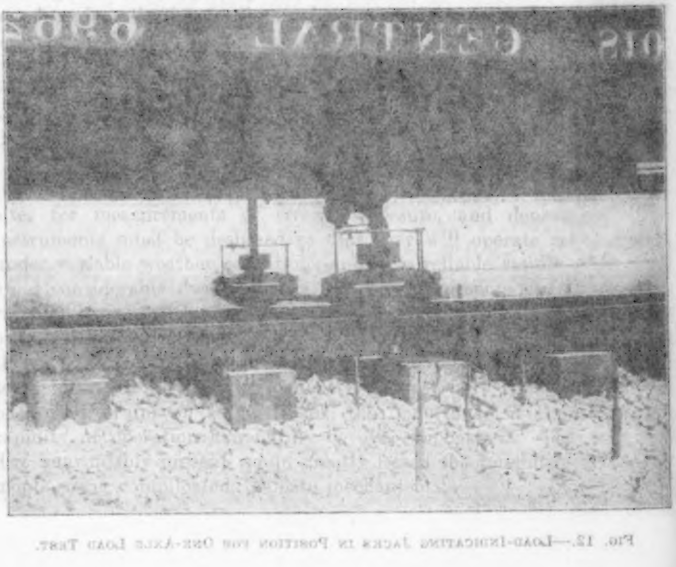


Fig. 12.—Load-Indicating Jacks in Position for One-Axle Load Test.

Nearly all the instruments used were designed especially for this work. To meet the severe conditions of field tests, they differ from the forms which might be suitable for those to be used under laboratory conditions. This difference is especially great for those used under rapidly moving loads. In many respects the problems involved in their design were new, and in some cases many attempts were made and a number of forms were tried out before satisfactory results were obtained. Much time was spent in experimenting with various forms of instruments to meet the needs of existing conditions. Details have been changed from time to time as improvements were devised. Those described herein show the final form chosen after the various modifications of details. In all cases they have given consistent results, have been found to be sufficiently sensitive to permit satisfactory interpretation of test results, have been accurate within the limits which were thought necessary and desirable, and have proved reliable and simple in operation.

9.—The Strain Gauge and Its Use in Measuring Strain in Rails Under Static Load.—In the tests with static loading, longitudinal strains in the rails under bending load were measured by means of a Berry strain gauge, Fig. 11. The strain gauge measures the change of distance between two small drilled holes, *A* and *B*, called the gauge points, which lie at the two ends of a gauge line along which the strain is to be measured. To the frame of the strain gauge are attached a dial micrometer and two pointed legs; one leg, *B*, is attached rigidly to the frame; the other leg is pivoted to the frame. The pivoted leg forms one arm of a bent lever, *A-C-D*, the other arm of which bears against the dial micrometer, *M*, which is attached to the frame of the instrument. The pointed ends of the legs of the instrument are placed in the gauge holes, and the reading of the micrometer dial is noted. This is done both with no load and with a given load. For any gauge line, the total strain is given by the difference between the reading under no load and the reading under load.* The general method of using this instrument consists in taking readings on a series of gauge lines with no load on the rail, and then taking a series of readings on the same gauge lines with a known load on the rail. For accurate work, this instrument requires the touch and care of a skilled operator. The unit strain is found by dividing the total strain by the length of the gauge line, and the longitudinal stress is determined by multiplying the unit strain by the modulus of elasticity of the steel, which, for the purposes of these tests, has been taken as 30 000 000 lb. per sq. in. The length of the gauge line used was 4 in. The gauge holes at the ends of each gauge line were drilled about $\frac{1}{16}$

* For a more detailed discussion of the strain gauge and its use, see *Bulletin 64*, Engineering Experiment Station, University of Illinois, "Tests of Reinforced Concrete Buildings under Load", by Talbot and Slater, and *Proceedings*, Am. Soc. for Testing Materials, 1915, p. 1019, "Use of the Strain Gauge in Testing Materials", by Slater and Moore.

in. deep with a No. 54 drill (0.055 in. in diameter) and were in the top of the base of the rail about ½ in. from the edge. (See Fig. 11.)

10.—The Level-Bar, and its Use in the Measurement of Rail Deflection and Tie Depression under Static Load.—For measuring the deflection of rail and the depression and bending of ties, the level-bar shown by Fig. 13 was used. It is a modification of an instrument used by Dr. P. H. Dudley, Consulting Engineer of the New York Central Lines, for determining tie and rail depressions, differing from Dr. Dudley's instrument in the length of bar and in the micrometer device. The distance between the two conical points, A and B, was 20 in.

In use, A is placed on a reference point, and B is placed on the spot for which the deflection is desired. The leveling nut, N, is then turned, raising or lowering the plunger, P, until the horizontal bar stands level, as shown by the level bubble. The dial micrometer, M, is attached to the plunger, P. A reading of the dial micrometer is taken without load and another with load. The difference between the two readings gives the change in elevation at one point with

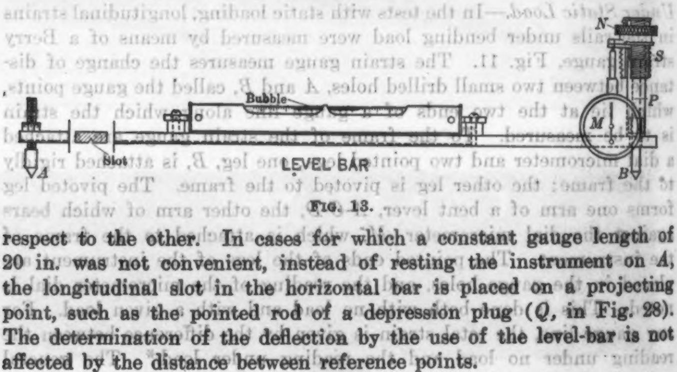


FIG. 13.

respect to the other. In cases for which a constant gauge length of 20 in. was not convenient, instead of resting the instrument on A, the longitudinal slot in the horizontal bar is placed on a projecting point, such as the pointed rod of a depression plug (Q, in Fig. 28). The determination of the deflection by the use of the level-bar is not affected by the distance between reference points.

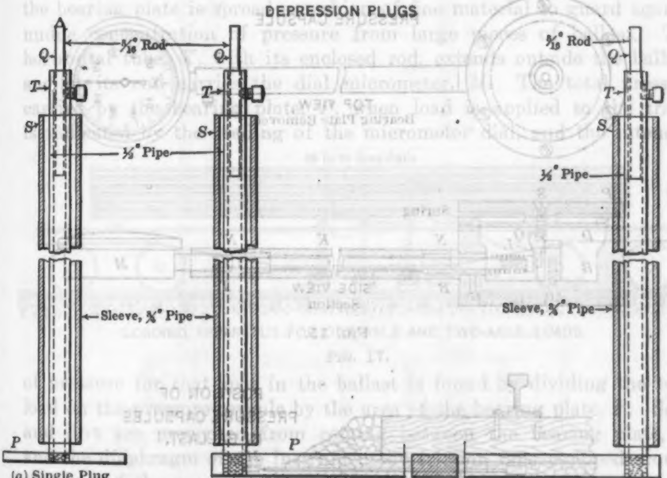
11.—The Depression-Plug, and its Use in the Measurement of Depression in Ballast and Roadway.—In order to measure the depression under static load of the ballast at various points and depths, and also the depression of the roadway, the device shown at the left in Fig. 14 was used. This device is called a depression-plug. It consists of a plate, P, about 3 in. square, to which is attached a tube, T, with a rod, Q, which is adjustable for height. The plate, P, is placed in the ballast, or over the roadway where the depression is to be measured, and the outer tube, S, is placed over the tube, T. Direct contact of T with the ballast would cause friction between them, and consequently the depression of P would be affected. The enclosing of T with S prevents this. The lower end of the outer tube, S, is raised an inch or two above the plate, P. As depression occurs at P, the rod, Q, is

depressed an equal distance, and the depression may be measured with the level-bar.

For measuring the depressions directly under a tie, the double depression-plug, shown at the right in Fig. 14, was used. The action of this plug is the same as that of the single depression-plug. The average of the depression, indicated by readings taken on both rods of the double plug, was taken as the average depression at the middle of the width of a tie.

12.—The Pressure-Capsule, and its Use in the Measurement of Pressure in Ballast.—For measuring the pressure transmitted to various parts of the ballast, the pressure-capsule shown by Fig. 15 was

DEPRESSION PLUGS



(a) Single Plug

(b) Double Plug

FIG. 14.

used. The elastic deflection of a thin steel diaphragm is used to measure the pressure applied to the capsule. The pressure to be measured is received on the bearing plate, *P*, which has an area of 5 sq. in.; the pressure is transmitted to the thin steel diaphragm, *D*, which is fastened by screws around its circumference to the cast-iron case, *B*. The screw, *S*, which fastens the plate, *P*, to the diaphragm, *D*, is hardened, and bears on one knife-edge of the small bell-crank lever, *L*, which is pivoted at *Q*. The vertical deflection of the center of the diaphragm is transmitted by the bell-crank lever (magnified about three times) in a horizontal direction to the rod, *R*, which slides in the guides, *NN'*, and is enclosed in a horizontal tube, *K*, and finally bears against the plunger, *T*, of an indicating dial micrometer, the

movement of which is thus a measure of the elastic deflection of the diaphragm, *D*. If the material of the diaphragm is not stressed beyond its elastic limit, the deflection of the diaphragm, and the consequent movement of the pointer of the dial micrometer, may be used to measure the load on the plate, *P*. Each pressure-capsule was calibrated by placing it on a platform scale and, with a screw clamp, applying a series of loads covering the range of its use. The instrument was recalibrated after use. The dial of the micrometer was arranged so as to be adjusted to read zero at zero load. This does away with the need of applying a correction for zero reading.

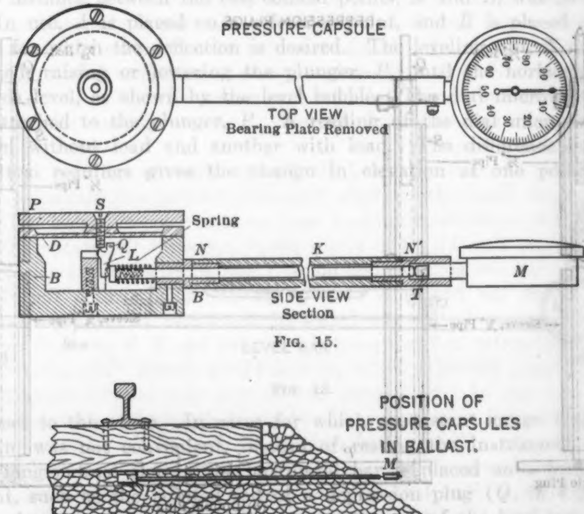


FIG. 15.

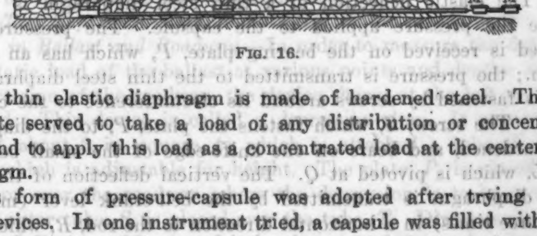


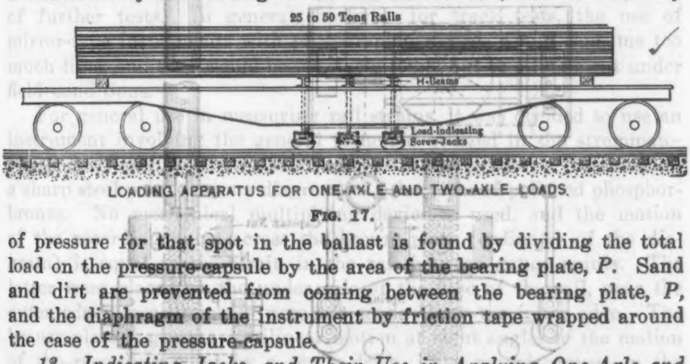
FIG. 16.

The thin elastic diaphragm is made of hardened steel. The bearing plate served to take a load of any distribution or concentration on it, and to apply this load as a concentrated load at the center of the diaphragm.

This form of pressure-capsule was adopted after trying various other devices. In one instrument tried, a capsule was filled with water, and the deflection of the cover was measured by the quantity of water forced into a small tube connected with the capsule. This instrument

proved to be inaccurate under varying temperatures. In another form, the deflection of the cover of the capsule acted on the ends of a slightly bent spring, and the lateral motion of the spring was measured. This form was less sensitive than that shown by Fig. 15. Attempts were made to measure pressures by the varying electrical conductivity produced in various substances by variations in pressure, but this did not give satisfactory results.

The position of the pressure-capsules as placed in the ballast is shown by Fig. 16. At the spot where it is desired to measure the pressure, a capsule is placed, with the surface of the bearing plate, P, perpendicular to the direction of the pressure to be measured. Above the bearing plate is spread a cushion of fine material to guard against undue concentration of pressure from large pieces of ballast. The horizontal tube, K, with its enclosed rod, extends outside the ballast, and at its end carries the dial micrometer, M. The total pressure carried by the bearing plate, P, when load is applied to the track, is indicated by the reading of the micrometer dial, and the intensity



LOADING APPARATUS FOR ONE-AXLE AND TWO-AXLE LOADS.

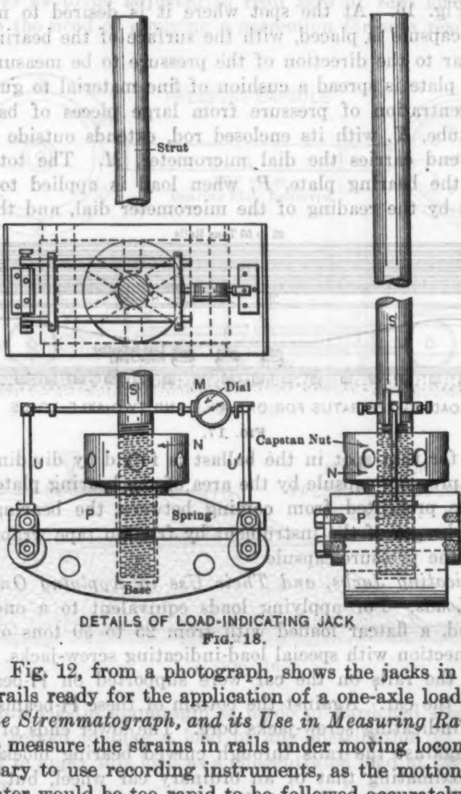
FIG. 17.

of pressure for that spot in the ballast is found by dividing the total load on the pressure-capsule by the area of the bearing plate, P. Sand and dirt are prevented from coming between the bearing plate, P, and the diaphragm of the instrument by friction tape wrapped around the case of the pressure-capsule.

13.—Indicating Jacks, and Their Use in Applying One-Axle and Two-Axle Loads.—For applying loads equivalent to a one-axle or a two-axle load, a flatcar loaded with from 25 to 50 tons of rails was used in connection with special load-indicating screw-jacks. As shown in Fig. 17, the rails on the car were supported on **H**-beams placed crosswise of the car. Against the bottom of these **H**-beams the upper ends of the indicating screw-jacks bore. The lower ends of the screw-jacks bore against the rails through curved bearing blocks having a radius approximating that of an ordinary car wheel, but not coned. On Fig. 17 the location of the indicating screw-jacks is shown in solid lines for the two-axle, and in broken lines for the one-axle, loading.

The construction of the load-indicating screw-jacks is shown in detail on Fig. 18. The load is applied by turning the capstan nut, N, on the screw, S, and the load is measured by the inclination given to

the ends of the flat nickel-steel spring, *P*. As this spring deflects under load, the ends are inclined, and with them the uprights, *U*, *U'*. The distance between the tops of the uprights is lessened, and the length of the motion is indicated by the dial micrometer, *M*. The load-indicating screw-jacks were calibrated in the laboratory by loading them with known loads in a testing machine. Stresses and depressions were generally measured for one rail only, but, in order to have both rails symmetrically loaded, it was necessary to use load-indicating jacks on



DETAILS OF LOAD-INDICATING JACK
FIG. 18.

both rails. Fig. 12, from a photograph, shows the jacks in place under the car of rails ready for the application of a one-axle load.

14.—The Stremmatograph, and its Use in Measuring Rail Strains.—In order to measure the strains in rails under moving locomotive loads, it is necessary to use recording instruments, as the motion of an indicating pointer would be too rapid to be followed accurately by the eye. Recording instruments for track tests under rapidly moving loads must withstand severe vibration, must be used in places exposed to dust and dirt, and must permit a large number of readings to be taken. It is evident that some simple form of recording device is essential,

one with few parts, one which may be quickly attached in position, and one giving a record which may be read quickly and accurately.

Various types of recording instruments were considered before deciding on the one to be used. Instruments with multiplying levers were regarded as unsuitable; experience with recording instruments has shown that uncertain effects are introduced by the inertia of moving parts, if the load is applied rapidly. Instruments involving the use of a ray of light reflected from a mirror to a sensitized photographic film were considered. This type gives the highest degree of sensitiveness of any considered, but its use would involve rather cumbersome arrangements of light-proof tubes leading from the mirror to the film; a special source of light would be necessary, and much time would be required to develop the films. A very ingenious mirror-type extensometer was designed and constructed by Mr. W. M. Dawley, and this instrument may be of great service in case very delicate measurements of strains are found to be necessary in the development of further tests. In general, however, for track tests, the use of mirror-type instruments with photographic records would consume too much time, and they would be too likely to get out of adjustment under field conditions.

For general use in measuring rail strains, it was decided to use an instrument involving the general principle utilized in the stremmatograph developed by Dr. P. H. Dudley. In Dr. Dudley's instrument*, a sharp steel point draws a diagram on a flat plate of polished phosphor-bronze. No mechanical multiplying device is used, and the motion of the recording point across the bronze plate (ordinates of the diagram) is equal to the strain in the rail between gauge points. The latter were placed on the under side of the base of the rail, near the outer edge; measurements were not made along the inner edge. The bronze plate is given a rectilinear motion at right angles to the motion of the recording point by a screw drive. The record graven on the bronze plate is examined under a high-power microscope, and the strain at any desired point on the record, given by the height of the diagram, is measured with a filar micrometer eye-piece.

The instrument used by the Committee differs in several important details from the stremmatograph described above. When the record is made on a bronze plate, the scribing needle must be re-ground after taking a few records. This re-grinding was a slow process, requiring a high degree of skill. After experimenting with various methods, it was decided to make the records of strain on a glass disk very lightly

* The following articles by Dr. P. H. Dudley give some results of tests made with his stremmatograph:

"Stresses in Rails under Moving Loads", *Railroad Gazette*, May 20th, 1898.
"Stresses in Railway Track under Moving Trains", *Engineering News*, Oct. 6th, 1898.
"Stresses in Rails under Moving Loads", *Railroad Gazette*, Oct. 21st, 1898.
"Rail Stresses—Tests of Unit Fiber Strains", *Engineering News*, Nov. 14th, 1901.
"Stremmatograph Tests, etc.", *Bulletin*, Int. Railway Congress, June, 1904.

smoked. It was found that a steel needle sweeps a smooth-edged, sharply-defined path through the very fine particles of lampblack which coat the disk, and that the width of such a path is very uniform.

After experimenting with various kinds of needles, it was found that a steel phonograph needle was suitable for making stremmatograph records on smoked glass, and they were inexpensive and could be readily obtained. Phonograph needles as purchased were found to be too hard and brittle, and, before being used, the temper was drawn slightly by heating them in oil at a temperature of 536° Fahr. (280° cent.) for 30 sec. and then quenching in water.

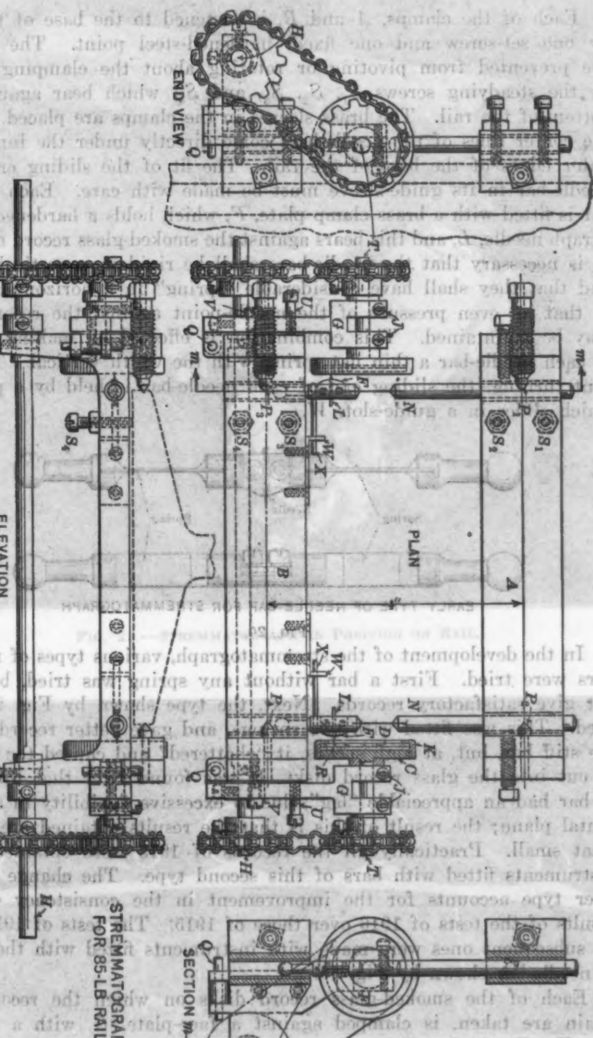
In order to give length to the record the disk was given a rotary motion, instead of the rectilinear motion used by Dr. Dudley, and a circular diagram was drawn, rather than a straight-line diagram. Freedom from play in bearings is of the very highest importance for securing accuracy in such an instrument. Bearings for circular motion can be made free from perceptible play more easily than can those for straight-line motion. The plane of the record disk is vertical. The most serious vibration of the rail is the up and down motion; this has the least dulling effect on the needles, and causes the least variation in the width of the line, if the plane of the record disk is vertical.

For smoking the glass disks used in the stremmatograph and other recording instruments, gasoline was used. Of all the various coatings tried, it was found that the smoke from burning gasoline gave one on which the needle scratches left the sharpest, clear-cut lines. In smoking the disks, they were held in a special device and passed back and forth through the smoke. The gasoline was burned in a small open cup holding about 2 oz.

As the tests progressed, various details of the stremmatograph were altered. The latest form of this instrument is shown by Fig. 19. Two clamps, *A* and *B*, are attached to the base of the rail, 4 in. apart, and between them extend two needle-bars, *NN*. One end of each needle-bar is in the shape of a spherical head with a slender stem; the other end is clamped by a set-screw to the clamp, *A* (called the anchor clamp); and the spherical end of each needle-bar is free to slide in a brass sleeve in the clamp, *B* (called the record clamp). Any point on the needle-bar will move with respect to any point on the record clamp, *B*, a distance equal to the stretch or the shortening of the rail along the line of the needle-bar. With the instrument shown, records are obtained for strains along both the inner and outer edges of the base of the rail.

In the first design, measurements of strain were made on only one side of the rail, but, when it was found that there was an appreciable difference between the stresses on the two sides of the rail, the design was altered so as to include measurements on the two sides.

Each of the clamps, A and B, is clamped to the base of the rail by one set-screw and one fixed hardened-steel point. The clamps are prevented from pivoting or rocking about the clamping points by the steadying screws, S_1, S_2, S_3 and S_4, which bear against the bottom of the rail. The [illegible] clamps are placed so that the [illegible] directly under the flange and [illegible] of the sliding end of a needle-bar in its guide [illegible] must be made with care. Each needle-bar is fitted with a brass clamp plate, F, which holds a hardened phonograph needle, L, and this bears against the smoked-glass record disk, D. It is necessary that the needle-bars be rigid in a vertical plane, and that they shall have considerable "spring" in a horizontal plane, so that an even pressure of the needle point against the record disk may be maintained. This combination is effected by making a part of each needle-bar a thin spring, W. [illegible] To prevent [illegible] the sliding end of the needle-bar is held by a pin, J, which [illegible] in a guide-slot [illegible]

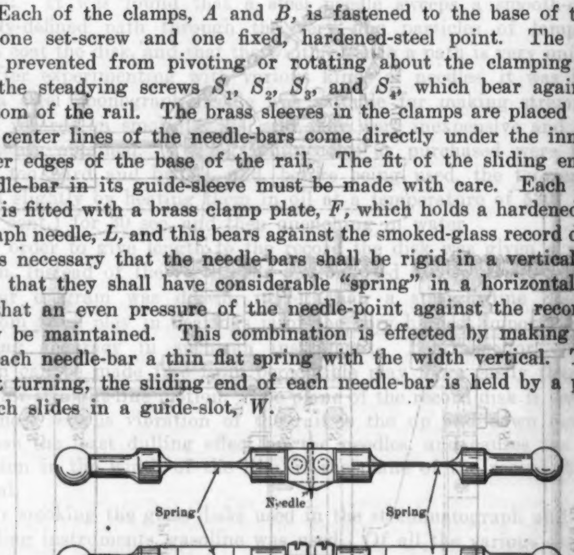


STREMMATOGRAPH FOR 85-LB RAIL.
Fig. 19.

In the development of the stremmatograph, various types of needle-bars were tried. First a bar without any spring was tried, but did not give satisfactory records. Next, the type shown by Fig. 20 was used. [illegible] and gave better records than the stiff bar, but, as [illegible] it [illegible] and [illegible] the needle to [illegible] the glass [illegible] type of bar had an appreciable [illegible] excessive flexibility in a horizontal plane; the results [illegible] the results obtained were somewhat small. Practically all the records of 1915 were taken with instruments fitted with bars of this second type. The change to the later type accounts for the improvement in the consistency of the results of the tests of 1916 over those of 1915. The tests of 1916 and all subsequent ones were made with instruments fitted with the form of needle-bar [illegible]

Each of the smoked-glass record disks on which the records of strain are taken, is clamped against a face-plate, C, with a screw-cap K (Fig. 19). Between the record disk and the face-plate is placed a circular piece of sheet-rubber, R. Each face-plate is driven by a shaft, T, which rotates in double bearings, G, and is itself driven by a sprocket and chain from a drive-shaft, H, below the instrument.

Each of the clamps, A and B, is fastened to the base of the rail by one set-screw and one fixed, hardened-steel point. The clamps are prevented from pivoting or rotating about the clamping points by the steadying screws S_1, S_2, S_3, and S_4, which bear against the bottom of the rail. The brass sleeves in the clamps are placed so that the center lines of the needle-bars come directly under the inner and outer edges of the base of the rail. The fit of the sliding end of a needle-bar in its guide-sleeve must be made with care. Each needle-bar is fitted with a brass clamp plate, F, which holds a hardened phonograph needle, L, and this bears against the smoked-glass record disk, D. It is necessary that the needle-bars shall be rigid in a vertical plane, and that they shall have considerable "spring" in a horizontal plane, so that an even pressure of the needle-point against the record disk may be maintained. This combination is effected by making a part of each needle-bar a thin flat spring with the width vertical. To prevent turning, the sliding end of each needle-bar is held by a pin, X, which slides in a guide-slot, W.



EARLY TYPE OF NEEDLE-BAR FOR STREMMATOGRAPH

FIG. 20.

In the development of the stremmatograph, various types of needle-bars were tried. First a bar without any spring was tried, but did not give satisfactory records. Next, the type shown by Fig. 20 was used. This was fitted with two springs, and gave better records than the stiff bar, but, at high speeds, it "chattered" and caused the needle to cut into the glass record disk. It was found later that this type of bar had an appreciable "lag", due to excessive flexibility in a horizontal plane; the result of this is that the results obtained are somewhat small. Practically all the records of 1915 were obtained with instruments fitted with bars of this second type. The change to the later type accounts for the improvement in the consistency of the results of the tests of 1916 over those of 1915. The tests of 1916 and all subsequent ones were made with instruments fitted with the form of needle-bar shown by Fig. 19.

Each of the smoked-glass record disks on which the records of strain are taken, is clamped against a face-plate, C, with a screw-cap K (Fig. 19). Between the record disk and the face-plate is placed a circular piece of sheet-rubber, R. Each face-plate is driven by a shaft, T, which rotates in double bearings, G, and is itself driven by a sprocket and chain from a drive-shaft, H, below the instrument.

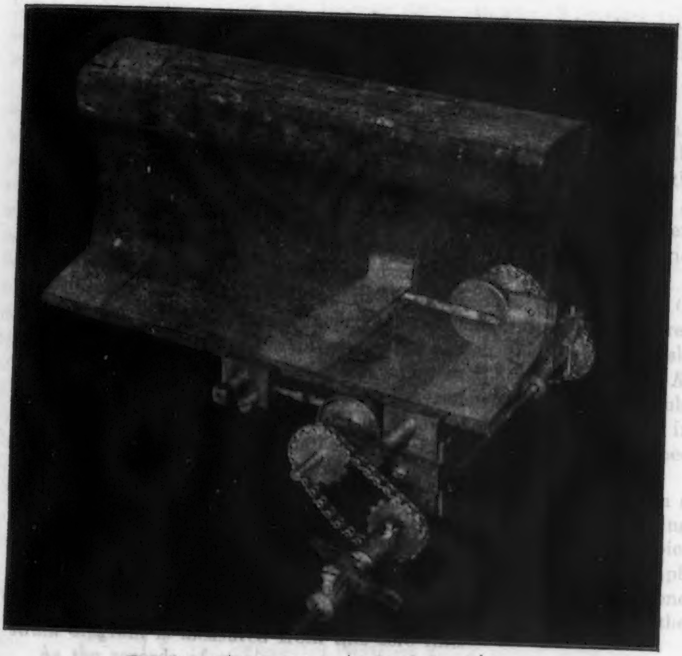


FIG. 21.—STREMMATOGRAPH IN POSITION ON RAIL.

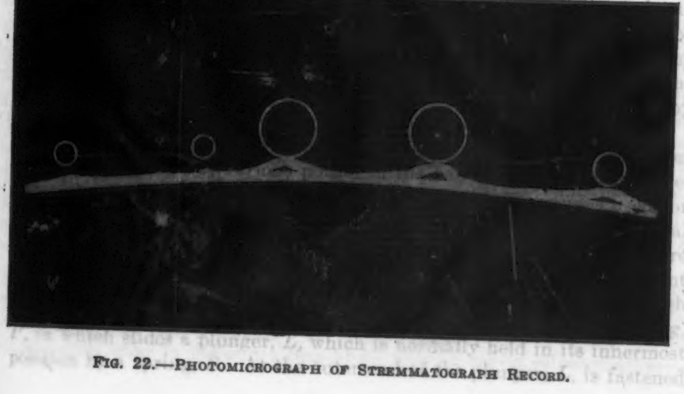


FIG. 22.—PHOTOMICROGRAPH OF STREMMATOGRAPH RECORD.

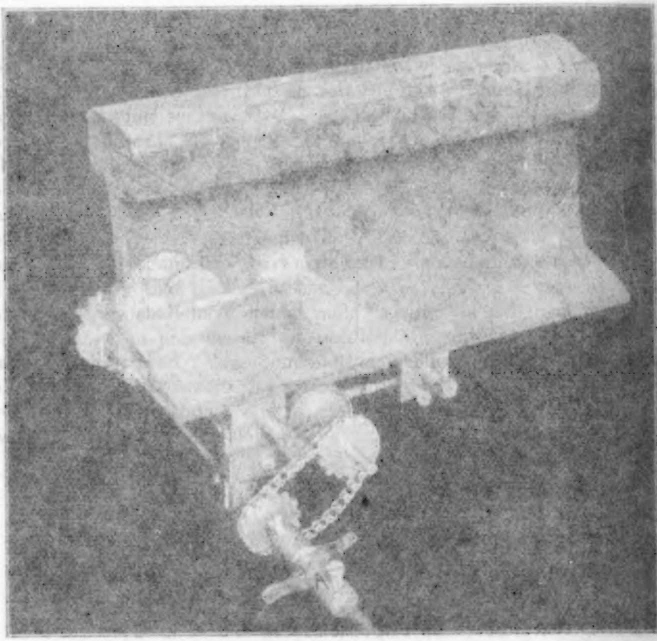


Fig. 21.—Stremmatograph in Position on Rail.

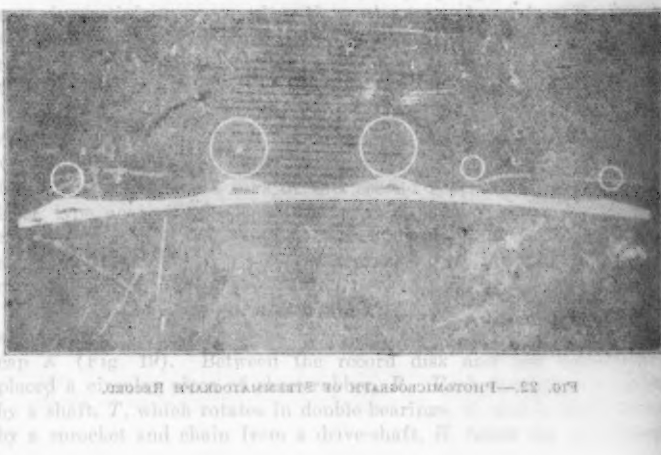


Fig. 22.—Photomicrograph of Stremmatograph Record.

This drive-shaft turns in eccentric bearings which permit adjustment of the chain tension, and is driven, through a flexible-spring coupling (shown at *A,* Fig. 21), by a long shaft which extends to one side of the track.

It is of highest importance that all play be taken up in the bearings of the drive-shaft, *T,* as any radial play is recorded as strain on the smoked-glass disk. The double bearings, *G,* were ground to fit the shaft, *T,* and the split bearings were fitted with tightening screws, *J,* for taking up wear. In order to eliminate all play, the bearings were tightened on the shaft so that the latter drove with considerable friction. Washers and collars were used to prevent axial play.

For convenience in changing disks or needles, the bearings, *G,* for the disk-shaft, *T,* were attached to the clamp, *B,* with a tapered pin, *Q,* which served as a pivot around which the bearings, disk-shaft, face-plate, and disk could be swung, so that the screw-cap, *K,* could be unscrewed, the smoked-glass disk removed, and another substituted, or so that the needle could be changed or adjusted. When in position for taking records, the bearings, shaft, and disk were clamped fast by the clamp screws, *U* and *V.*

Fig. 21 is from a photograph of a stremmatograph in position on a rail. That part of the instrument behind the rail is shown by making a double exposure of the photographic plate used in taking the picture. Fig. 22 is from photo-micrographs of typical stremmatograph records. The zero line was obtained by giving the record disk one complete revolution with no load on the track. The height of the strain diagram is measured from this zero line.

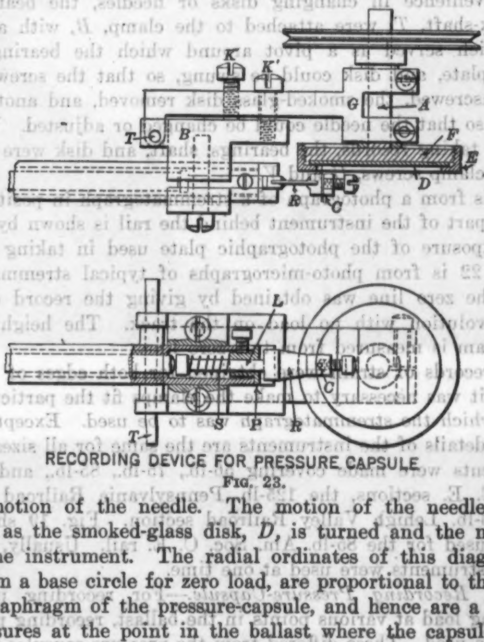
As the records of strain were obtained for both edges of the base of the rail, it was necessary to make the clamps fit the particular size of rail on which the stremmatograph was to be used. Except for the clamps, the details of the instruments are the same for all sizes of rail.

Instruments were made covering 56-lb., 75-lb., 85-lb., and 100-lb., Am. Soc. C. E. sections, the 125-lb. Pennsylvania Railroad section, and the 136-lb. Lehigh Valley Railroad section. Fig. 19 shows the instrument used for the 85-lb. Am. Soc. C. E. rail. Usually, for any test, four instruments were used at one time.

15.—The Recording Pressure-Capsule.—For recording pressures under moving load at various points in the ballast, recording pressure-capsules were used. These differed from the pressure-capsules used for static tests only in the substitution of recording devices for the dial micrometers used with the pressure-capsules for static loads, which are shown by Fig. 15. Fig. 23 shows the recording device for the instrument for tests with moving loads. In the end of the horizontal tube which extends sidewise from the pressure-capsule is fastened a bearing plug, *P,* in which slides a plunger, *L,* which is normally held in its innermost position by a spring, *S.* At the outer end of the plunger, *L,* is fastened

a flat spring, *R*, with its width vertical, and at the outer end of this flat spring is a needle-chuck, *C*, in which a needle is held by a set-screw.

A block, *B*, is clamped on the outer end of the horizontal pipe extending from the pressure-capsule. To this block is fastened, by a taper pin, *T*, and clamping screws, *K K′* a double bearing, *G*, in which turns a shaft, *A*, to which is attached a face-plate, *F*, on which a smoked-glass disk, *D*, is clamped by the screw-cap, *E*. The bearing, the face-plate, the disk, and the screw-cap are the same as the corresponding pieces used with the stremmatograph. For the recording pressure-capsule, the disk shaft, *A*, is driven by a round belt operating the grooved pulley, *Y*. The deflection of the diaphragm of the pressure-capsule causes a motion of the plunger, *L*, which, in turn,



RECORDING DEVICE FOR PRESSURE CAPSULE
FIG. 23.

causes motion of the needle. The motion of the needle draws the diagram as the smoked-glass disk, *D*, is turned and the moving load passes the instrument. The radial ordinates of this diagram, measured from a base circle for zero load, are proportional to the deflection of the diaphragm of the pressure-capsule, and hence are a measure of the pressures at the point in the ballast where the capsule is placed. The measurement of the radial ordinates of pressure-capsule records is made with a microscope fitted with a micrometer eye-piece.

When the pressure-capsule was being designed, little attempt was made to work out the details of an instrument which would operate satisfactorily under moving loads, as only static-load tests were being made. Later, the recording device was designed for use with the pressure-capsules under moving loads, and a number of tests were run.

It is thought, however, that, due to the inertia of parts in the instruments, measurements of pressures under moving loads at speed will be inaccurate and unreliable, especially at high speeds.

16.—Drive Rig for Moving-Load Tests.—The drive apparatus for turning the disks of the recording instruments in the moving-load tests is shown at one side of the track in Fig. 25. The main drive-shaft is driven by hand, through a crank and worm reducing gear, and this, in turn, drives small shafts which are connected with the several stremmatographs. Although the attempt was made to have the speed of rotation of the drive about proportional to the speed of the locomotive, this was not considered a very important matter. As long as the drive was fast enough to separate the portions of the curve corresponding to the passage of successive wheels, no trouble was encountered in reading the various strains from the curves, nor was it necessary to have a means for establishing the position of the locomotive with respect to the location of the instrument, other than that given by the records themselves. As can be seen from Fig. 22, the records show definite maximum values in the curves corresponding to the passage over the instruments of various wheels of a locomotive. It is obvious that such a maximum value can only occur when a wheel is over the instrument, so that it is unnecessary to have other means for determining the position of the locomotive.

17.—Measurement of Deflection of Rail with a Camera.—In moving-load tests, the deflection of various points of the rail was measured by using a double-exposure photograph. At intervals there were glued along the outside of the rail small pieces of black paper with small white crosses on them. A Speed Graphic camera for 5 by 7-in. plates was set up at a distance of about 10 ft. from the rail, as shown by Fig. 25. At this distance the vibration of the camera was found to be small, and it did not affect the results of the measurement materially, except at high speeds. The camera was fitted with an anastigmat lens having a speed of F 4.5 and a focal-plane shutter giving a minimum length of exposure of 0.001 sec. For any given observation, a photograph was made of the unloaded rail with its attached white crosses. Then, without moving the camera or changing the plate, the camera shutter was set for the next exposure. The camera shutter was fitted with an electro-magnet release, and the wires from the magnet led through a battery to a contact piece attached to the rail. As the locomotive ran on the test section of track the front truck, as it passed over the contact piece, closed the circuit through the electro-magnet, and a second photograph of the rail, with its attached white crosses, was made on the plate in the camera. On the plate, when developed, each small white cross shows as a vertical line crossed by two horizontal lines, and the distance between the horizontal lines is propor-

tional to the depression of the rail. This distance was measured with a microscope having a micrometer eye-piece, and multiplication by a factor gave the rail depression.

With one camera the depression of the rail may be measured for a length of about 10 ft. Generally, two cameras were used, both operated at the same time by the magnetic release. In examining the developed photographic plates with a microscope, the maximum magnification used was about 75 times. Under higher magnification, the silver grains on the sensitized surface of the plate showed as small particles to such an extent that the edges of the lines of which the distance apart was to be measured were blurred. With the camera 10 ft. from the track, and a magnification of 75 in the measuring microscope, the deflection of the rail can be determined with a precision of about 0.01 in.

For this photographic measurement, Cramer Crown, single-coated, photographic plates were used. These plates were found to combine high sensitiveness to light with fine grain of silver. Fig. 24 is from a print of a double-exposed plate for depression measurement.

18.—Measurement of Speed of Locomotive.—For measuring the speed of the locomotive as it passed the test section of track, an automobile speedometer of the revolving-magnet type was used. The speedometer was driven, through a flexible shaft, by a leather-faced friction wheel which ran on the tread of a wheel of the locomotive, generally the trailer. The speedometer was placed so that its dial was visible to the engineman and to an observer stationed in the cab. The accuracy of the speedometer was checked at various speeds by noting its reading while the locomotive was run at various uniform rates of speed over a stretch of track, the time being measured by a stop-watch. The friction-drive wheel of the speedometer was pivoted, so that it could be swung back out of contact with the wheel when the locomotive was run backward. When in operating position, the friction-drive wheel was held against the wheel by a spring.

19.—Manufacture of Instruments.—The strain gauge, the cameras for recording depression of track, the microscopes for reading records from recording instruments, and the speedometer, were purchased. The load-indicating jacks and the depression plugs were made by local firms from designs furnished. All other instruments were made in the shop of the Laboratory of Applied Mechanics of the University of Illinois by the mechanician employed on the test work. In all, there were manufactured 14 stremmatographs, 2 level-bars, 73 pressure-capsules, and various special appliances and connecting parts for apparatus. It should be stated that, after the instruments had been first made, it was necessary for the mechanician to spend considerable time in maintaining the instruments and in making such modifications as were found to promise better results.

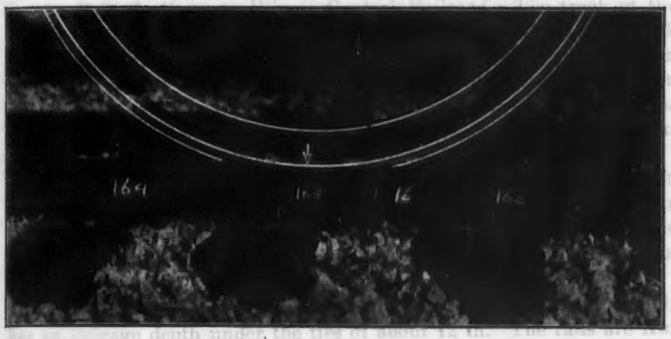


Fig. 24.—Double-Exposure Photograph, as Used in Measuring Track Depression.

Fig. 25.—Test Section on Illinois Central Railroad.

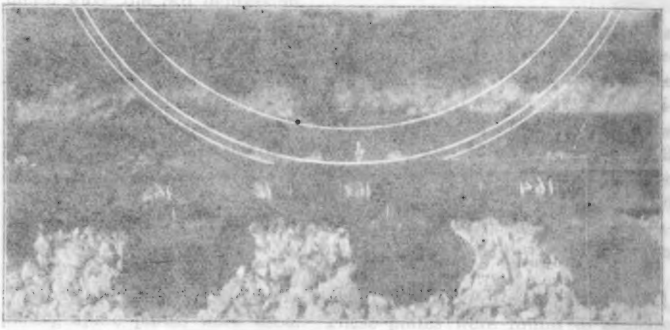


FIG. 24.—DOUBLE-EXPOSURE PHOTOGRAPH, AS USED IN MEASURING TRACK DEPRESSION.

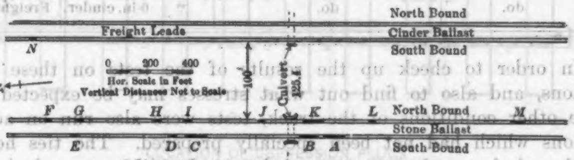
FIG. 25.—TEST SECTION ON ILLINOIS CENTRAL RAILROAD.

B.—Preparation of Test Track, and Procedure of Tests.

20.—Test Sections on Illinois Central Railroad.—The track of the Illinois Central Railroad used in the test work is on the double-track main line, about 2 miles north of Champaign, Ill. At this location all freight trains are diverted from the main-line tracks, and run through the yards over special freight tracks, only passenger trains being run over the main line. For test purposes, therefore, these tracks are comparatively free from traffic disturbances.

The stretch of track used is on an embankment, from 4 to 8 ft. high, composed of loam and clay. A single-track road was built in 1854, and the second track was added in 1900. Age has given compactness to the embankment, and it was in dry condition throughout the tests.

The ballast at this place consists of crushed limestone; it usually has an average depth under the ties of about 12 in. The rails are Am. Soc. C. E. section, 85 lb. per yd. The rails on the south-bound track are 33 ft. long, and were laid in 1902; those on the north-bound track are 30 ft. long, and were laid in 1900.



RELATIVE LOCATION OF TEST SECTIONS ON ILLINOIS CENTRAL RAILROAD

FIG. 26.

To provide uniform known conditions of track for the tests, four stretches were specially prepared. For these test sections special oak ties replaced the original ties. Four such sections were prepared within a short distance of each other. On one section the ballast had a depth of 6 in. below the ties; on another, 24 in.; and on two others, 12 in. On one of the last-named sections, the ties were 7 by 9 in. by 8 ft.; on all others they were 6 by 8 in. by 8 ft. In locating the test sections, depths of ballast closely approximating those desired were found, and the track was raised to make the proper depth. The special ties had been prepared accurately to size, and were of uniform quality. When first prepared, these special test sections were laid with the 85-lb. rail which had been in the track originally. Later, when it was desired to use heavier rails in the tests, these were removed and replaced by the heavier ones the sections of which are shown on Fig. 107. The sections used were chosen because they were readily available, and give considerable range in weight. The relative location of the various test sections is shown in Fig. 26. The size of rails used is

given in Table 1. The 100-lb. rails were placed on Section *K*, which has 24 in. of ballast and 6 by 8-in. by 8-ft. ties. The 125-lb. rails were placed on Sections *B* and *K*, which have 12 and 24 in. of ballast, respectively, and 6 by 8-in. by 8-ft. ties.

TABLE 1.—DESCRIPTION OF TEST SECTIONS ON ILLINOIS CENTRAL RAILROAD.

Section.	Rail.	Ties.	Ballast.	Remarks.
A	85 lb.	Sp'l. 7 by 9 in. by 8 ft. 0 in.	12-in. stone.	Special section.
B	85 and 125-lb.	Sp'l. 6 by 8 in. by 8 ft. 0 in.	12-in. "	do.
C	85-lb.	Sp'l. 6 by 8 in. by 8 ft. 0 in.	6 in. "	do.
D	do.	About 6 by 8 in. by 8 ft. 0 in.	About 6-in. stone.	Ordinary track.
E	do.	do.	do	Decayed tie.
F	do.	do.	do.	do.
G	do.	do.	do.	do.
H	do.	do.	do.	Tie-plates used.
I	do.	do.	do.	Decayed tie.
J	do.	do.	About 24-in. stone.	Ordinary track.
K	85, 100 and 125-lb.	Sp'l. 6 by 8 in. by 8 ft. 0 in.	24-in. stone.	Special section.
L	85-lb.	About 6 by 8 in. by 8 ft. 0 in.	About 15-in. stone.	Decayed tie.
M	do.	do.	" 8-in. "	Low spot.
N	do.	do.	" 6-in. cinder.	Freight lead.

In order to check up the results of the tests on these special sections, and also to find out what stresses may be expected under some other conditions of the track, tests were also run on adjacent sections which had not been specially prepared. The ties here are mostly oak, but replacements are being made with creosoted pine with tie-plates. The relation of the location of these sections to the special test sections will be seen from Fig. 26. Sections *D* and *J* were chosen as being representative track, in ordinarily good condition, and having the usual ties and tie spacing. Sections *E, F, G, I*, and *L* were chosen as showing the results of a decayed or badly cut tie. Section *M* was at a low spot in the track. Sections *D, E, F*, and *G* were used for static tests with the loading apparatus, and Sections *D, J, I, L*, and *M* were used for tests with moving loads. Section *H*, having tie-plates, was used for static tests. Section *N*, on the freight lead, having the ordinary run of ties and cinder ballast, was used for tests with moving loads. All this track was laid with 85-lb. rail. The depths of ballast were as indicated in Table 1.

21.—Preparation of the Test Sections.—In placing the ties used for the special test sections, the old ties were removed and the new ones put in without disturbing the ballast below the bottoms of the ties. This was done by the regular section men of the Illinois Central Railroad. For each test section, the depth of ballast was determined by excavating to sub-grade, near the ends of the ties on each side of the track at two points in each rail length. The track was raised suffi-

ciently to give the required depth of ballast under the ties, and in no case was it necessary to raise the track more than 2 in.

At the north rail length of each test section, the ties were spaced 22 in. from center to center. The purpose of this was to get three tie spaces for a 66-in. wheel spacing. Under the other two rail lengths of each section the ties were placed as nearly as possible according to the standard practice of the Illinois Central, which uses 18 ties to a 30-ft. rail and 20 ties to a 33-ft. rail (approximately 20-in. spacing). These ties were placed with considerable accuracy, the position that each was to occupy being marked on the rail for the guidance of the section men.

Whenever the track had been in use long enough to need it, it was tamped and put in good surface. After such resurfacing, tests were not run until sufficient time had elapsed for traffic to compact the ballast which had been disturbed; generally 10 days or 2 weeks were allowed.

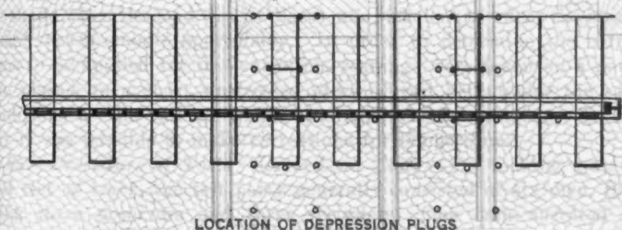
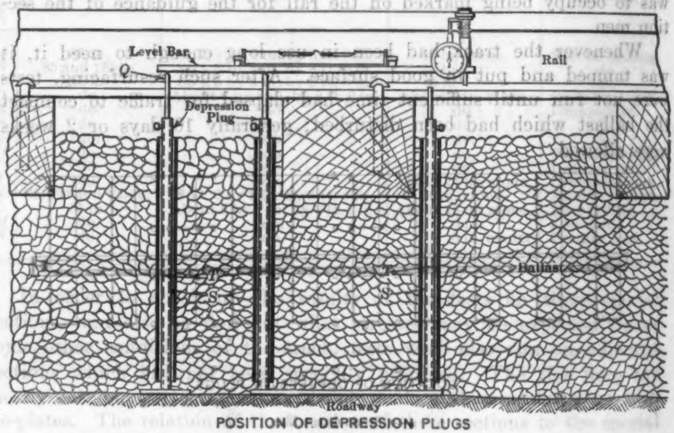


FIG. 27.

In preparing the special test sections, the depression-plugs were put in after the special ties had been placed and the track raised to its proper level. In putting in these plugs, both between and under the ties, the tie was removed and the ballast excavated down to the level of the sub-grade. The depression-plugs were then placed in the positions shown by Figs. 27 and 28, and the ballast was replaced. As this material was replaced it was carefully tamped. In placing certain of the single depression-plugs in the 24-in. ballast, a slightly different method was used. A piece of 6-in. pipe was driven with a sledge until its lower end was at the bottom of the ballast. The ballast inside the pipe was excavated, the depression-plug was put in, and the ballast was then replaced and thoroughly tamped as it was added. The pipe was then pulled up and used in placing the next depression-plug. After the depression-plugs had been in place a few days, the track was again thoroughly tamped. The track was re-tamped as often as the spot became low, until the tests were started. Although it was realized that this method of placing the depression-plugs was open to the objection

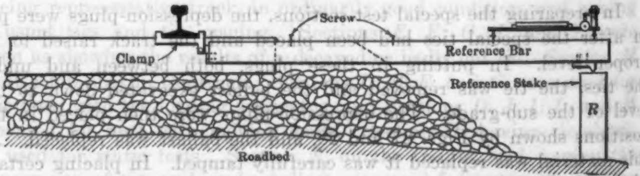
that it disturbed the ballast, there was no way of placing them without doing this. As no tests were made until the depression-plugs had been in place about 2 weeks, it is thought that the ballast was fairly well compacted by that time.

After the depression-plugs were in place, the rods (Q, Fig. 28) were put in and adjusted for height so that the level-bar could be used from the rail to the adjacent plugs, and from these plugs to the others. On Fig. 12 the projecting upper ends of the tubes and rods of several depression-plugs can be seen.



POSITION OF DEPRESSION PLUGS

FIG. 28.



ARRANGEMENT OF APPARATUS FOR MEASURING TRACK DEPRESSION

FIG. 29.

22.—Preparation of Test Track for Level-Bar Measurements.—To furnish a fixed reference point from which measurements of depression could be made, a reference stake, R, was driven 80 in. from the rail for which the deflection was to be measured (Fig. 29). A reference bar, consisting of a piece of 2 by 4-in. lumber, was fastened at one end with a pivoting joint by a clamp to the rail, the other end resting on a knife-edge-bearing on the reference stake. Level-bar measurements were made from the reference stake to the rail, readings

being taken between screws in the top of the reference bar, the outer screw being directly over the knife-edge. From the point on the rail readings were taken along the rail and from the rail to the various depression-plugs. As shown by Fig. 29, readings were taken from the reference stake to the rail and then along the rail. Additional reference stakes were placed at sufficiently close intervals to furnish a check on the depression readings.

23.—Preparation for Strain-Gauge Measurements.—For a static test, preparations were made for the strain-gauge measurement of longitudinal strains in the rail by drilling gauge holes at the ends of the gauge line along which strain was to be measured. These gauge holes were on the top of the base of the rail, about ¼ in. from its edge, and, in the early tests, on only one side of the rail. These gauge lines were placed over every tie and between ties for the three rail lengths of each test section. Fig. 27 shows the typical arrangement of gauge lines for a test section of track.

It was found that the change in temperature of the rail (while in the shade when the load was in place) was sufficient to affect the strain-gauge measurements appreciably. In order to determine the corrections to be applied for this variation, readings were taken on a gauge line placed on a short unstressed piece of rail kept under the same conditions as the rail on which readings were being taken. In this way it was possible to make corrections for temperature.

24.—Procedure of Tests; Static-Load Tests.—For making a static-load test of track, the test party generally consisted of six men, there being three observers and three recorders. One man operated the strain gauge, one handled the level-bar along the rail, and another handled the level-bar on the depression-plugs. Each of these observers had his own recorder.

The typical procedure of the static-load tests, when the loading apparatus for one-axle and two-axle load was used, was as follows: The car, loaded with rails, was taken to the test section and carefully "spotted" so that the H-beams (Fig. 17) came directly over the points where the load was to be applied. The brakes of the car were then set and the engine was uncoupled and run away from the test section far enough not to affect the results. After the load-indicating jacks had been put in place, ready to apply the load, zero-load readings were taken with the strain-gauge and the level-bars. Load was then applied with the jacks, and a set of load readings was taken. The next load increment was applied and the load readings taken. After the desired number of increments of load had been applied (usually four), the load was removed and the zero-load readings were again taken. Readings on the standard gauge lines were taken at the beginning and end of each set of load and zero-load readings. To take a complete set of readings, including two sets of zero readings and four load readings,

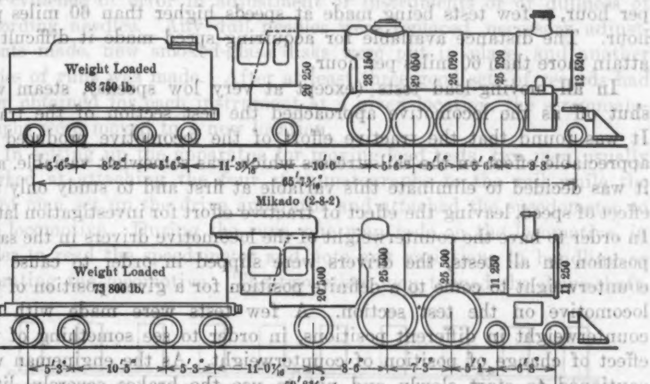
required from 1½ to 2 hours. For the one-axle load, there were a total of about 80 strain-gauge readings, 115 level-bar readings along the rail, and 90 lever-bar readings on the depression-plugs. For the two-axle load, there were about 115 strain-gauge readings, 115 level-bar readings along the rail, and 125 level-bar readings on the depression-plugs.

The typical procedure in static-load tests with a locomotive was practically the same as when the loading apparatus was used. Zero-load readings were taken with all instruments. The locomotive was then run on the test section and spotted at the desired point. Load readings were taken, and then the locomotive was spotted at a new position and load readings taken. After taking load readings with the locomotive at the number of points desired, the locomotive was run off the test section and zero-load readings were again taken. As it required about 1 hour to take a set of readings under the full length of the locomotive and tender, it was thought best to check up on the zero-load readings at frequent intervals, in order that errors introduced into strain-gauge results by the variations of temperature might be corrected. Hence, when readings were taken under the full length of the locomotive, generally one set of load readings was taken between the two sets of zero-load readings. Each set of readings under the full length of the locomotive and tender included about 120 strain-gauge readings, 60 level-bar readings to the rail, and 30 level-bar readings to the depression-plugs. As many of these depression-plugs were under the locomotive, the 30 readings on these required about as long as the 60 readings on the rail.

25.—Procedure in Measuring Static Pressure in Ballast.—The measurement of the pressures in the ballast with the pressure-capsule generally was not made at the same time as the other tests. In measuring these pressures, the dial micrometers were placed in the ends of all the tubes of the pressure-capsules, and the dials were set to read zero. Load was applied at the desired point, either by the loading apparatus or the locomotive, and the dials were then read. As soon as the desired loads or load in the desired position had been applied and the readings taken, the load was removed, and the zero-load readings were checked to see if they returned to zero. These readings under load were reduced to unit pressures, expressed in pounds per square inch, as described later.

26.—Procedure of Tests; Moving-Load Tests.—Tests in which the load on the test track was produced by a locomotive running over the section are designated as moving-load tests. On the tests on the Illinois Central Railroad, three types of locomotives were used in moving-load tests, a Mikado (2-8-2), an Atlantic (4-4-2), and a Pacific (4-6-2). Fig. 30 gives diagrams of the locomotives used, with their wheel loads. The same types were used on static-load as on moving-

load tests, except that the switching locomotive was used only on static-load tests and the Pacific locomotive only on moving-load tests. In all tests with the Atlantic type, the same locomotive was used. The Pacific locomotive used in the 1916 tests was not the same as that used



DIAGRAMS OF
FOUR TYPES OF ILLINOIS CENTRAL LOCOMOTIVES.
FIG. 30.

in those of 1915. In tests with the Mikado type, use was made of whatever Mikado locomotive was available; and, in the course of the work, tests were made with 25 of this type. In all cases except the one where the equalizer was found blocked up, the locomotives were in good working condition. The tires were in good condition, there being in

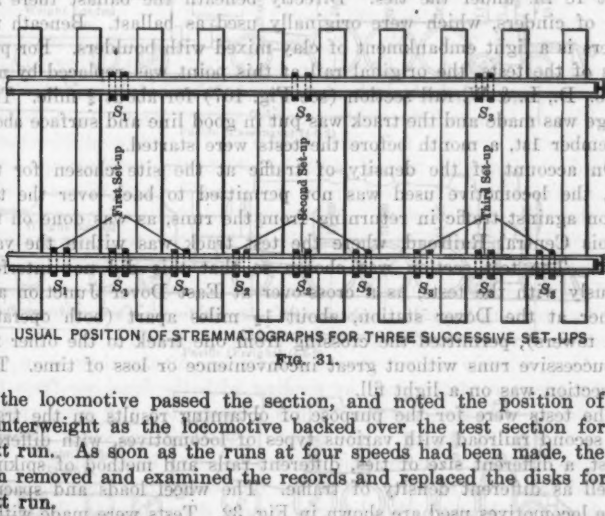
no case evidence of much wear. The Mikado locomotives were used at speeds up to 35 miles per hour, that being the maximum speed permitted by the regulations of the Illinois Central Railroad. The Atlantic and Pacific locomotives were used at speeds up to 60 miles per hour, a few tests being made at speeds higher than 60 miles per hour. The distance available for acquiring speed made it difficult to attain more than 60 miles per hour.

In all moving-load tests (except at very low speeds), steam was shut off as the locomotive approached the test section of the track. It was found that the tractive effort of the locomotive produced an appreciable effect on the rail stresses which was somewhat variable, and it was decided to eliminate this variable at first and to study only the effect of speed, leaving the effect of tractive effort for investigation later. In order to have the counterweight of the locomotive drivers in the same position in all tests, the drivers were slipped in order to cause the counterweight to come to a definite position for a given position of the locomotive on the test section. A few tests were made with the counterweight in different positions, in order to see something of the effect of change of position of counterweight. As the engineman was cautioned to start slowly and not to use the brakes severely, little trouble was experienced from the position of the counterweight changing during the tests.

For moving-load tests, the party usually consisted of three men. The procedure of a typical moving-load test was as follows: The stremmatographs were attached to the rails, and the recording devices were attached to the pressure-capsule tubes; then the drive-shaft rigging was set up and connected to the stremmatographs and to the recording pressure-capsules. The speedometer was attached to the locomotive, and, in case it was desired to measure rail deflection, the cameras for measuring deflection were set up and the electrical connections for tripping the camera shutters made. Smoked-glass disks were then put in place on the stremmatographs and on the recording pressure-capsules, the pressure of the disks against the recording needles was adjusted, and a zero-load line was drawn on the disks by rotating the drive-shaft. The locomotive was then run over the test section at the lowest speed used (5 miles per hour), and records were obtained by turning the drive-shaft by the hand-wheel. Then the locomotive was run over the test section at next to the highest speed used (25 miles per hour for the Mikado, and 50 for the Atlantic and Pacific locomotives). A run was then made at next to the lowest speed (15 miles per hour for the Mikado and 35 for the Atlantic and Pacific locomotives), and, last, a run was made at the highest speed (35 miles per hour for the Mikado, and 60 for the Atlantic and Pacific locomotives). Although this order and these speeds were not always followed, they are typical. The records for the runs at four speeds

were obtained on the same glass disks. After the runs at four speeds, the smoked-glass disks were removed and examined under a microscope for evidence of error in adjustment of instruments or of dullness of recording needles. Any dull needles were replaced, necessary adjustments made, new smoked-glass disks were put in place, and another series of runs was made. After at least three good sets of records had been obtained for each instrument at a given location, the stremmatographs were moved to a new location.

In setting up the apparatus for moving-load tests, two men usually worked at attaching the four stremmatographs to the rail while the third man set up the drive apparatus and attached the speedometer to the locomotive. During the runs, one man rode on the locomotive, in order to read the speedometer and note any variations in handling or operating the locomotive. The two other men turned the drive-shaft,



USUAL POSITION OF STREMMATOGRAPHS FOR THREE SUCCESSIVE SET-UPS

FIG. 31.

as the locomotive passed the section, and noted the position of the counterweight as the locomotive backed over the test section for the next run. As soon as the runs at four speeds had been made, the two men removed and examined the records and replaced the disks for the next run.

27.—*Preparation for Moving-Load Tests.*—Fig. 25 shows the arrangement of the apparatus as set up for the moving-load tests, and Fig. 31 is a diagram of the usual arrangement of stremmatographs. Four stremmatographs were used at one time, three on one rail between adjacent ties, and one on the other rail opposite the middle instrument of the three. At each test section the instruments were set up successively at three adjacent locations, covering a distance of eight tie spaces near the center of the rail length. Strain measurements were made along the middle of the rail, rather than near the ends,

because the effect of rail joints would be appreciable near the ends, and this effect was regarded as a special problem to be studied later.

Under favorable conditions, after skill had been acquired, the necessary number of records for a given weight of rail and one condition of track, and for one type of locomotive, were obtained in from 1½ to 2 days.

28.—Tests on D., L. & W. R. R.—Tests were made on the tracks of the Delaware, Lackawanna and Western Railroad at a point near Dover, N. J., during the fall of 1916.

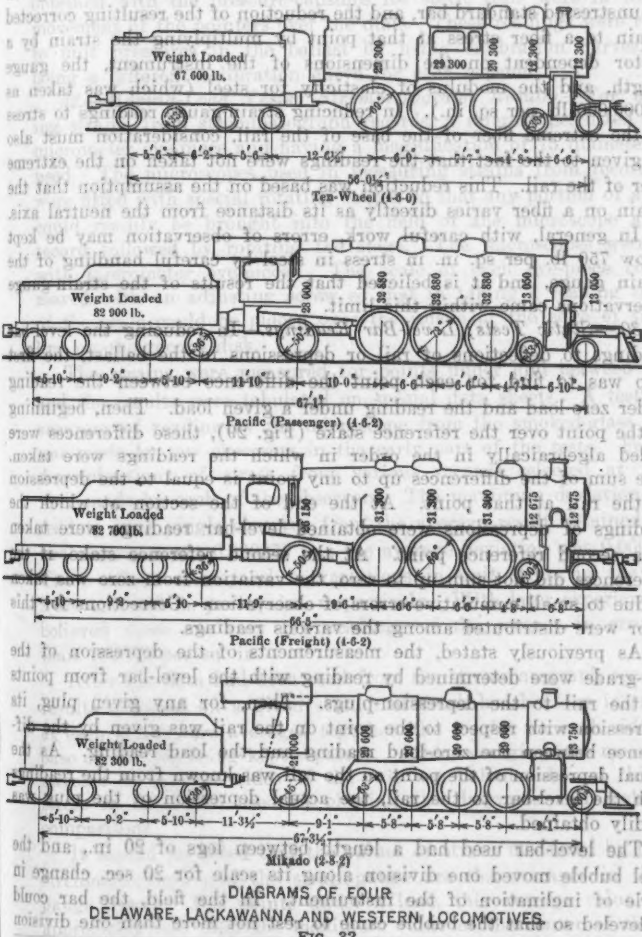
The section of track chosen was on the east-bound main line, about one mile east of the station at Dover. The tracks here are laid with 101-lb. D., L. & W. R. R. rails on 7 by 9-in. by 8 ft. 6-in. creosoted pine ties, tie-plates being used on every tie and screw-spikes throughout. The ballast consists of trap rock having a depth of about 18 in. under the ties. Directly beneath the ballast there are 2 ft. of cinders, which were originally used as ballast. Beneath the cinders is a light embankment of clay mixed with boulders. For purposes of the tests, the original rail at this point was replaced by new 105-lb., D., L. & W. rail section (see Fig. 107) for about ½ mile. This change was made and the track was put in good line and surface about September 1st, a month before the tests were started.

On account of the density of traffic at the site chosen for the tests, the locomotive used was not permitted to back over the test section against traffic in returning from the runs, as was done on the Illinois Central Railroad, where the test track was within the yard limits. The test section was chosen so that this did not interfere seriously with the tests, as a cross-over at East Dover Junction and another at the Dover station, about 1½ miles apart (both operated from towers), permitted the crossing from one track to the other for the successive runs without great inconvenience or loss of time. The test section was on a light fill.

The tests were for the purpose of obtaining results on the track of a second railroad with various types of locomotives, with different ballast, a different size of ties, different rails and method of spiking, as well as different density of traffic. The wheel loads and spacing of the locomotives used are shown in Fig. 32. Tests were made with a Ten-wheel, a Mikado, and two Pacific locomotives. Of the two Pacific locomotives used, one is for passenger service and the other for fast freight service. The method of making the tests was the same as that followed on the Illinois Central Railroad, described elsewhere.

C.—Reduction of Data, and Accuracy of Instruments.

29.—Static Tests; Strain-Gauge Readings.—In static tests, the strain-gauge readings, taken on the top of the base of the rails, were reduced to stresses in the extreme fiber of the base of the rail. The



DIAGRAMS OF FOUR
DELAWARE, LACKAWANNA AND WESTERN LOCOMOTIVES.

FIG. 32.

general process of reduction of a strain-gauge reading* involved the subtraction of the reading under load from the reading under zero-load, the correction of this difference by reference to the readings on an unstressed standard bar, and the reduction of the resulting corrected strain to a fiber stress at that point by multiplying the strain by a factor dependent on the dimensions of the instrument, the gauge length, and the modulus of elasticity for steel (which was taken as 30 000 000 lb. per sq. in.). In reducing strain-gauge readings to stress in the extreme fiber of the base of the rail, consideration must also be given to the fact that the readings were not taken on the extreme fiber of the rail. This reduction was based on the assumption that the strain on a fiber varies directly as its distance from the neutral axis.

In general, with careful work, errors of observation may be kept below 750 lb. per sq. in. in stress in steel by careful handling of the strain gauge, and it is believed that the results of the strain-gauge observations came within this limit.

30.—Static Tests; Level-Bar Readings.—In reducing the level-bar readings to deflections of rail or depressions in the ballast, the first step was to find for each point the difference between the reading under zero-load and the reading under a given load. Then, beginning at the point over the reference stake (Fig. 29), these differences were added algebraically in the order in which the readings were taken. The sum of the differences up to any point is equal to the depression of the rail at that point. At the end of the section at which the readings of depressions were obtained level-bar readings were taken to a second reference point. At the second reference stake, if the differences did not sum up to zero, the variation from zero was taken as due to small cumulative errors of observation. Corrections for this error were distributed among the various readings.

As previously stated, the measurements of the depression of the sub-grade were determined by reading with the level-bar from points on the rail to the depression-plugs. Then, for any given plug, its depression with respect to the point on the rail was given by the difference between the zero-load reading and the load reading. As the actual depression of the point on the rail was known from the readings with the level-bar to the rail, the actual depression of the plug was readily obtained.

The level-bar used had a length between legs of 20 in., and the level bubble moved one division along its scale for 20 sec. change in angle of inclination of the instrument. In the field, the bar could be leveled so that the bubble came to rest not more than one division

* For a detailed discussion of the method used in reducing the data see *Bulletin No. 64*, Engineering Experiment Station, University of Illinois, "Tests of Reinforced Concrete Buildings under Load," by A. N. Talbot and W. A. Slater; also, see *Proceedings*, Am. Soc. for Testing Materials, 1913, "The Use of the Strain Gauge in Testing Materials," by W. A. Slater, and H. F. Moore. The accuracy of strain-gauge readings is discussed in the latter paper.

from the middle of its scale, and this corresponds to a change of elevation of one end of the bar with respect to the other of 0.002 in.

31.—Static Tests; Pressure-Capsule Readings.—The readings obtained with the pressure-capsules for static tests were in terms of movement of the pointers over the dials. These readings were reduced to unit pressures in the ballast by using calibration curves, there being a different calibration curve for each capsule.

32.—Moving-Load Tests; Stremmatograph Results.—For measuring strains from stremmatograph records, a microscope, fitted with a micrometer eye-piece and having a magnification of 75 diameters, was used. The microscopes used in measuring strains from record disks were fitted with special rotating stages, so that any portion of a record could readily be brought into the field of the microscope and the record followed by rotating the stage. The microscopes were fitted with micrometer eye-pieces. The scale of each eye-piece could be moved with an adjusting screw so that in taking a reading the zero of the scale could be placed over the image of the edge of the zero line on the record disk.

The strains were measured for points under and between wheels, and the results were tabulated on special data sheets. A test of the accuracy of reading by the microscope from the smoked-glass records was made by two observers in the early part of the tests. Forty-five determinations of stress from various records selected at random were made by each of the observers. The maximum deviation of any reading by a single observer from the average result obtained by the two observers was 0.000270 in.; the average deviation was 0.000054 in. These values correspond to variations in unit stress in the rail of 2 020 and 400 lb. per sq. in., respectively. With later experience, it is believed there are very few readings of the records in error more than 700 lb. per sq. in.

After the data had been read from the stremmatograph records, all results obtained on a given section of track by the four instruments at each speed were averaged together, the results for each side of the base of the rail being kept separate. After averaging, the mean of the stresses at the two sides of the rail was calculated, and this mean stress is the value used in plotting the diagrams and making the comparisons.

It should be noted that the values of strain measured by both the stremmatograph and the strain-gauge are the averages in the 4-in. gauge length. For points under a load, this average value may be appreciably less than the maximum value at the center of the gauge line, so a correction has been applied to obtain the maximum stress. In the static-load tests in which the strain-gauge was used, this correction was computed as follows: If f_0 is the measured stress under

the load, and f_1 is the measured stress at the adjacent gauge line at a distance x (in inches) from the load, then it can be readily shown that the value to be added to f_0, in order to obtain the stress at the center of the gauge line under the load, is given by the following equation:

$$\text{Correction} = \frac{f_0 - f_1}{x - 1}$$

For the stremmatograph results, no readings at adjacent gauge lines were available for use in making this correction. From the static tests, however, it was found that this correction averaged practically 4% of the average stress under the load. Therefore, all the observed stresses under wheels in the moving-load tests were increased by 4% in order to obtain the stress at the center of the gauge line.

33.—Moving-Load Tests; Results with Recording Pressure-Capsules.—The records obtained with the recording pressure-capsules were of the same general character as those obtained with the stremmatographs. In reducing these results, readings with the microscope were taken of the values of the ordinates of the diagrams under the wheels and between the wheels of the locomotive, as was done with the stremmatograph records. By multiplying by the proper factor, these readings were reduced to movements of the plunger rods (expressed in thousandths of an inch). These quantities were then used to obtain the unit-pressure in the ballast from the calibration curves. After being computed and checked, these pressures were grouped and averaged, as was done with the stremmatograph results.

34.—Moving-Load Tests; Photographic Measurement of Depression.—The notes taken in connection with the photographic measurement of rail depression and tie depression included: description of test section, speed of locomotive, condition of light, time of day, the size of the stop (diaphragm) used, the length of exposure, the distance of the camera lens from the rail, and the position of the electric contact point for operating the shutter. The reduction of the data was made by microscope measurements of the distance between the images on the developed negative of the two positions of the reference cross-marks on the rail or on the ties. The reading of the scale in the micrometer eye-piece was reduced to inches of depression of the rail or tie by the use of a constant depending on the distance of the camera lens from the rail, the focal length of the lens, the scale value of the micrometer eye-piece, and the magnification used. From a comparison of results obtained by this method in static tests with results obtained by the use of the level-bar, measurements of deflection with the camera apparently can be made with an error of observation not greater than 0.01 in., except at high speed or on embankment subject to much vibration. The tests on the Illinois Central Railroad, however, were made on an embankment, and for that reason considerable inconsistency was found in the tests at speed.

IV.—Results of Tests.

35.—Form of Presentation.—In reporting the results of the tests, the effort has been made to present only those matters which seem to have a bearing on the fundamentals of track action and the problems of track, and to give the essential results, free as far as possible from the mass of details of the test data. When it is stated that the tests have involved the making, reading, recording, and reducing of more than 250 000 observations on rail strains alone, the need for presenting only the essentials will be apparent. Generally speaking, only averages of a considerable number of values are presented. Some use of individual results will be made in the discussion on the variation of individual values from the average value. It has been thought best to present the results largely in graphical form. This method allows general comparison to be made readily. Tabular values are also given for some of the principal results.

The results presented in this report relate principally to stresses in rail and to the depression of track as a whole. The action of the tie, the transmission of pressure through ballast and roadway, and other related matters must be reserved for a later report.

Data relating to depression of track under one-axle load, two-axle load, and locomotive loading under static conditions will first be presented, then data on stresses in rail for these loadings and for moving-load tests with locomotives; and, following these, a general discussion of the effect of speed, influence of rail section, effect of wheel spacing, condition of track, etc.

36.—Depression of Track Under Load.—Flexibility, elasticity, and stiffness are important properties of railroad track. The quality of the track is affected by variations in these properties. It is apparent even to a casual observer that track depresses under wheel loads. The weight from the wheel loads is distributed by the rail among adjacent ties, and vertical pressures are set up in ties, ballast, and roadway. The pressures transmitted by rail, ties, ballast, and roadway compress or otherwise deform these various parts of the track structure, the vertical deformation and movement of the different parts together forming the total track depression. Generally speaking, the action has the nature of elastic deformation, and, when the wheel load is removed, the track resumes its normal position, wholly or partly, according to the condition of the track and the nature and weight of the load. The stiffness and flexibility of track are dependent on the section of the rail and its flexural properties, the dimensions and spacing of the ties, and the nature, quality, and condition of the ballast and roadway.

At this place no effort will be made to analyze the parts played by the rail, ties, ballast, and roadway in making up the track depression.

The total depressions will be reported—the combined effect of rail, tie, ballast and roadway. At another time a discussion will be made of the relation between the deformations of the several parts of the track structure. The following general statement is given as an estimate of the division of the depressions in the various parts, in what may be called good track, under the drivers of a Mikado locomotive: compression of the tie under the rail and effect of bending of tie to bring it to full bearing on the ballast along its length, 0.05 in.; compression of 24 in. of stone ballast immediately under the rail, 0.15 in.; compression of roadway immediately under rail, 0.15 in. The bending of the rail between the ties is slight, the deflection of the rail between two adjacent ties under the weight of the driver of a Mikado locomotive on 85-lb. rail amounting to not more than 0.01 in. in a tie spacing of 22 in. For heavier rails, the deflection, of course, will be less.

Whether the magnitude of the track depression is directly proportional to the load applied, or varies considerably from direct proportionality, is dependent on the nature of the track and its condition. For the best track in well-tamped condition (freshly surfaced), the tests indicate that the relation between the load and the resulting depression approaches direct proportionality; that is, the depression of the track is directly proportional to the load applied. For mediocre track and track which is not well kept up, it is evident from the results of the tests that the first part of the load applied produces a greater depression than a later equal additional increment of load. The effect of this on stresses in the track is important.

The foregoing refers to the relation between vertical pressures and the resulting vertical deformations or depressions at any point along the line of the rail, both at and away from the wheel load. Away from the load, the pressure of the rail on the tie will vary from tie to tie, and therefore the consequent depression will vary, but, at any point, the downward pressure of the rail and the upward pressure of the tie must, of course, be equal to each other. In the tests, the magnitude of the track depression along the rail has been measured, and the distribution of vertical pressure among the ties may be estimated from the data obtained. Records of results will be given of track depression for one-axle load, two-axle load, and locomotive loading.

37.—Depression of Track; One-Axle Load and Two-Axle Load.—In Figs. 33 to 41 are given track depression profiles for one-axle and two-axle loads (the axles being 66 in. apart), for tests made on the test sections of track on the Illinois Central Railroad. At the place where the tests with loading apparatus were made, the ties were 22 in. from center to center. The load was applied near the middle of the length of a rail, in order to avoid the effect of rail joints. The measurements of depression were taken on the base of the rail; the depressions

reported give the vertical movement of the rail as produced by bending of rail and vertical movement of tie, ballast, and roadway. Results are given for the load applied both at a point directly over the tie and at a point between the two ties. Most of the tests are on rails of 85-lb. section, but results are also given for tests on 100-lb. and 125-lb. sec-

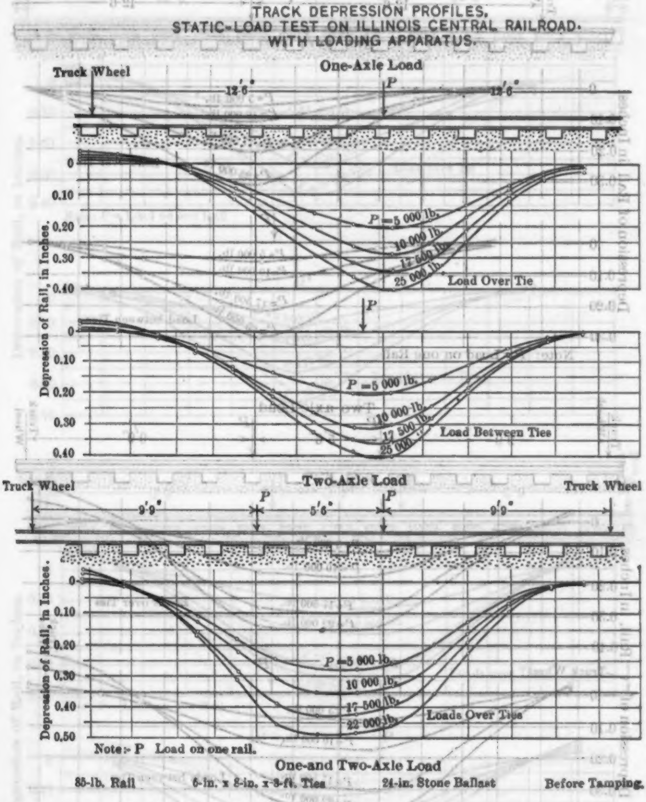
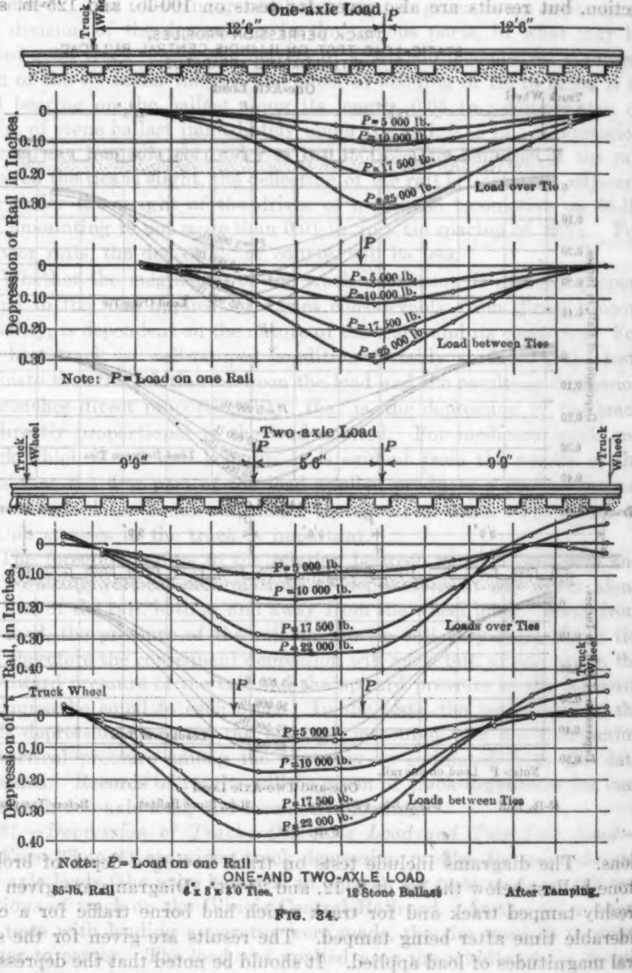


FIG. 33.

tions. The diagrams include tests on track having a depth of broken stone ballast below the tie of 6, 12, and 24 in. Diagrams are given for freshly-tamped track and for track which had borne traffic for a considerable time after being tamped. The results are given for the several magnitudes of load applied. It should be noted that the depression measurements were taken from the initial position which the track

reported give the vertical movement of the rail as produced by bending of rail and vertical movement of tie, ballast, and roadway. Results are given for the load placed directly over the tie and at a point between the ties. These tests are on rails of 85-lb. section, but results are also given for tests on 100-lb. and 125-lb. section.

TRACK-DEPRESSION PROFILES.
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD
WITH LOADING APPARATUS.

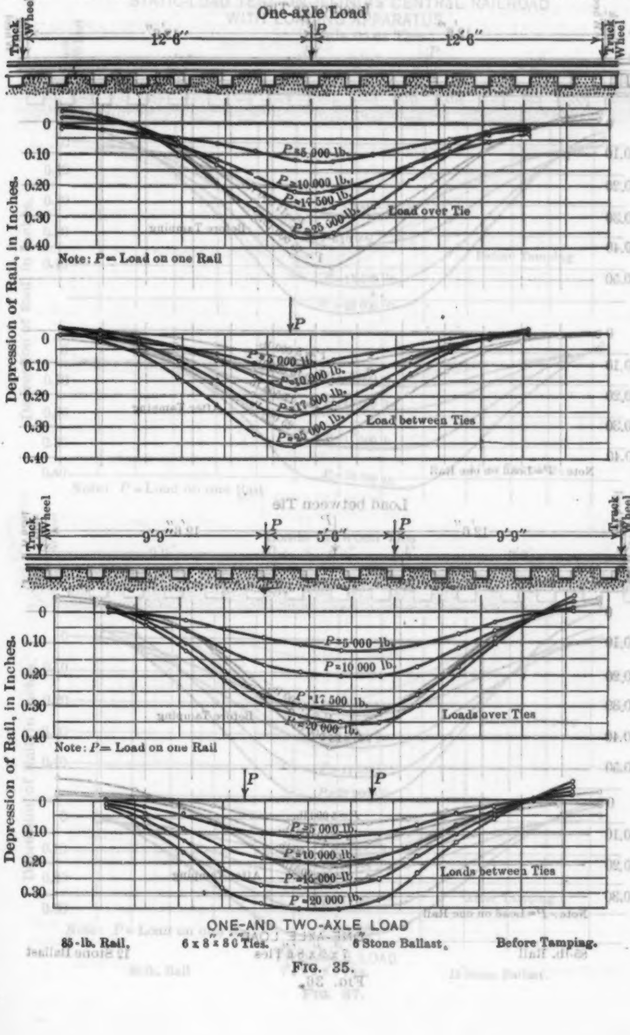


Note: P=Load on one Rail
ONE-AND TWO-AXLE LOAD
85-lb. Rail 6"x 8"x 8'0" Ties. 12" Stone Ballast After Tamping.

FIG. 34.

tions. The diagrams include tests on track of broken stone below the tie, 12, and diagrams are given for freshly tamped track and for track which had borne traffic for a considerable time after being tamped. The results are given for the several magnitudes of load applied. It should be noted that the depression measurements were taken from the initial position which the track

TRACK DEPRESSION PROFILES,
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS



85-lb. Rail. 6 x 8" x 8' 0" Ties. 6" Stone Ballast. Before Tamping.

FIG. 35.

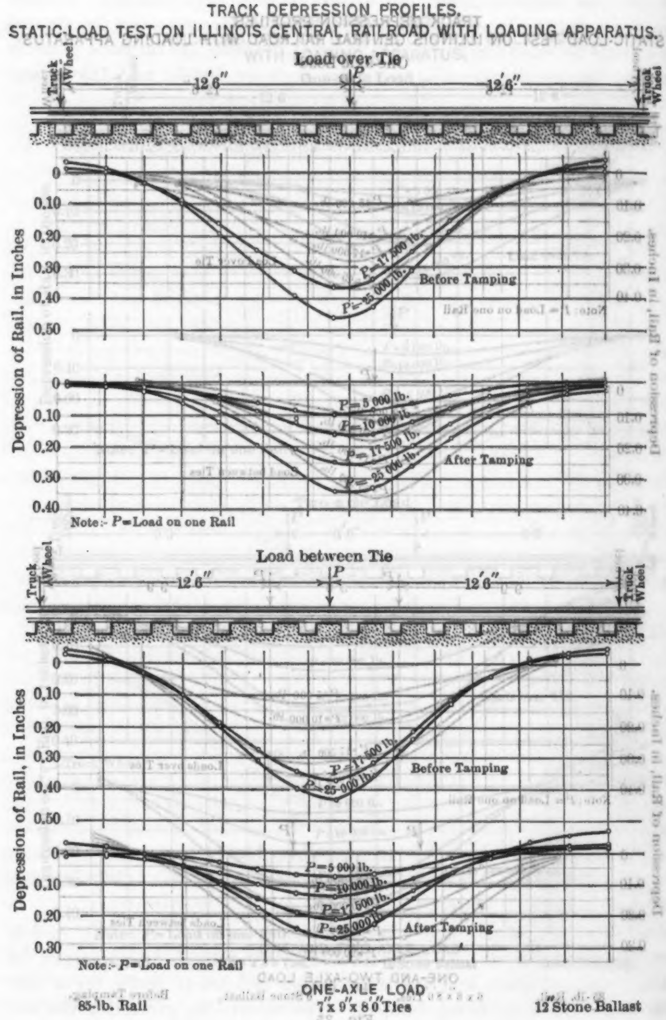

TRACK DEPRESSION PROFILES.
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.
Load over Tie
Truck Wheel
12′6″
P
12′6″
Truck Wheel
Depression of Rail, in Inches
0
0.10
0.20
0.30
0.40
0.50
P=17 500 lb.
P=25 000 lb.
Before Tamping
0
0.10
0.20
0.30
0.40
P=5 000 lb.
P=10 000 lb.
P=17 500 lb.
P=25 000 lb.
After Tamping
Note:- P=Load on one Rail
Load between Tie
Truck Wheel
12′6″
P
12′6″
Truck Wheel
Depression of Rail, in Inches
0
0.10
0.20
0.30
0.40
0.50
P=17 500 lb.
P=25 000 lb.
Before Tamping
0
0.10
0.20
0.30
P=5 000 lb.
P=10 000 lb.
P=17 500 lb.
P=25 000 lb.
After Tamping
Note:- P=Load on one Rail
ONE-AXLE LOAD
85-lb. Rail
7″x 9″x 8′0″ Ties
12″ Stone Ballast

Fig. 36.

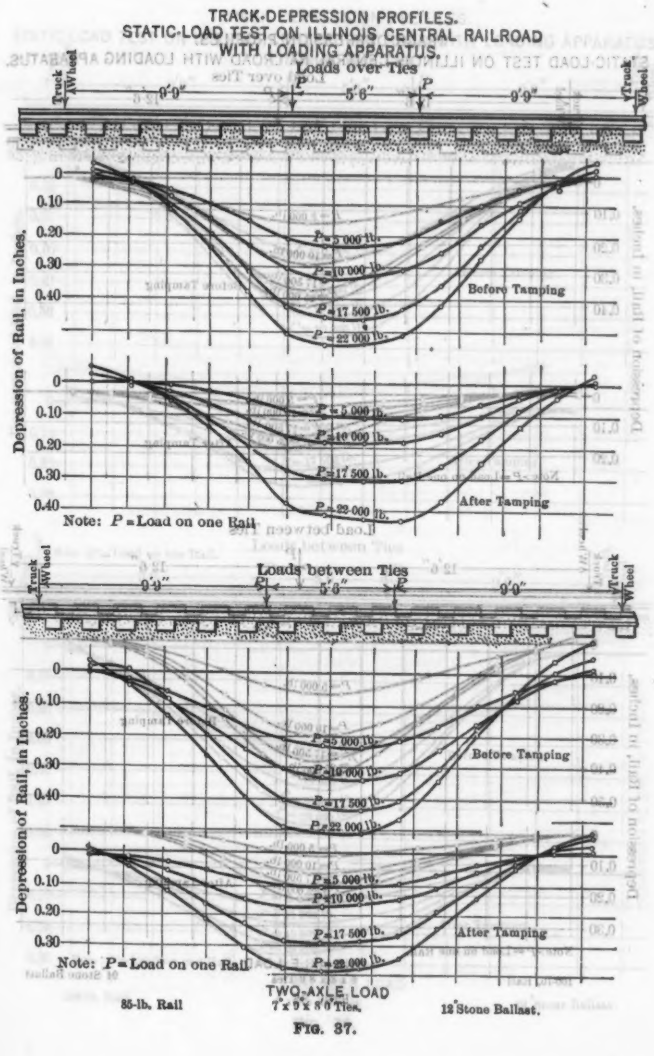
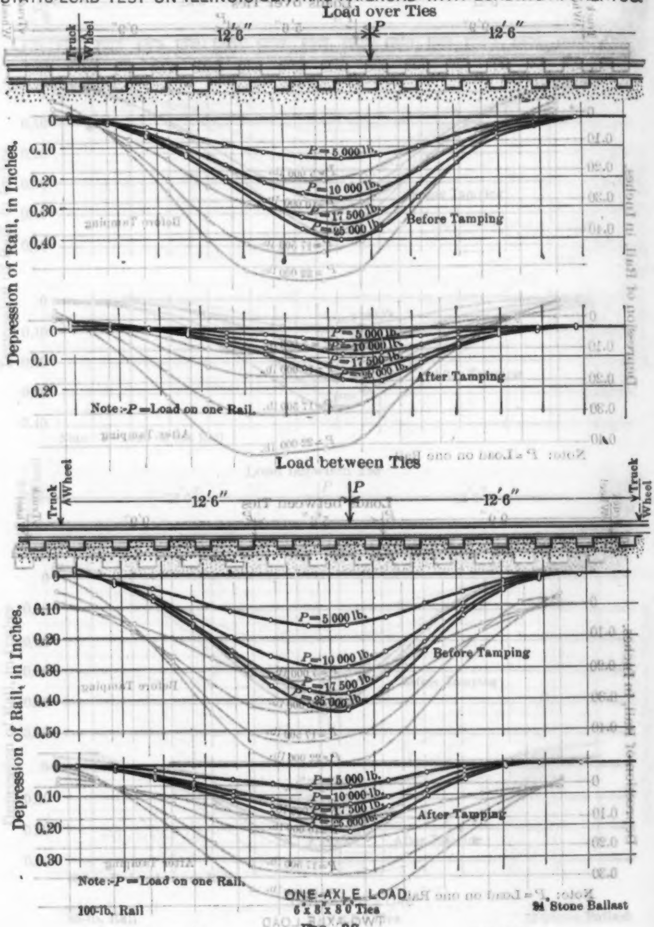

TRACK-DEPRESSION PROFILES.
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.
Loads over Ties
Truck Wheel
9'9"
P
5'6"
P
9'9"
Truck Wheel
Depression of Rail, in Inches.
0
0.10
0.20
0.30
0.40
P=5 000 lb.
P=10 000 lb.
P=17 500 lb.
P=22 000 lb.
Before Tamping
After Tamping
Note: P = Load on one Rail
Loads between Ties
TWO-AXLE LOAD
85-lb. Rail
7"x 9"x 8'0" Ties.
12" Stone Ballast.

FIG. 37.

TRACK DEPRESSION PROFILES.

STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.



ONE-AXLE LOAD.

100-lb. Rail 6"x 8"x 8'0" Ties 24" Stone Ballast

FIG. 38.

TRACK DEPRESSION PROFILES.

STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

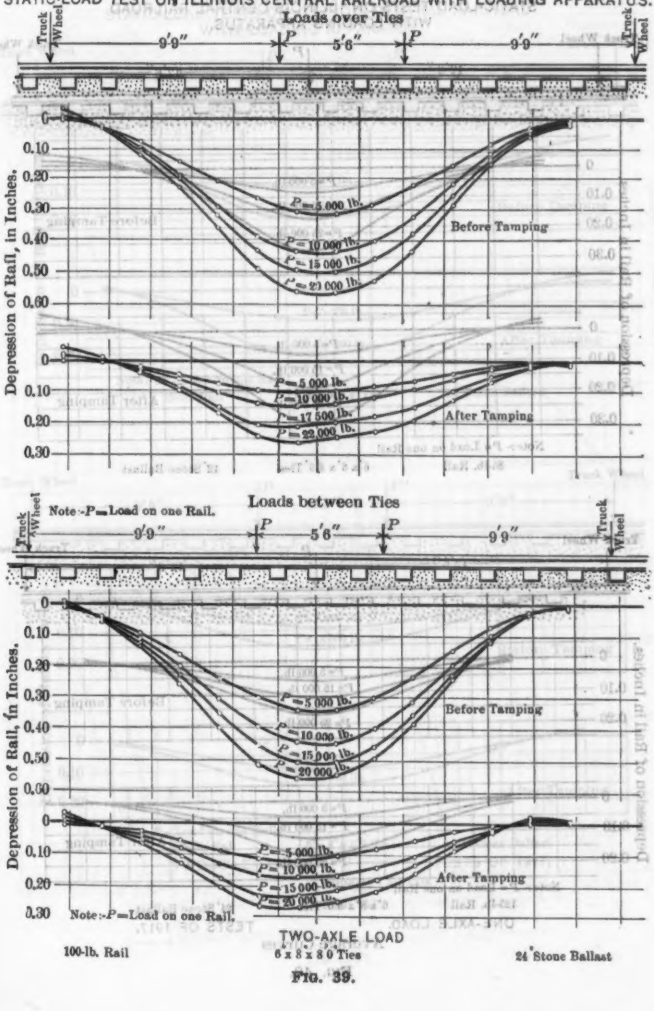
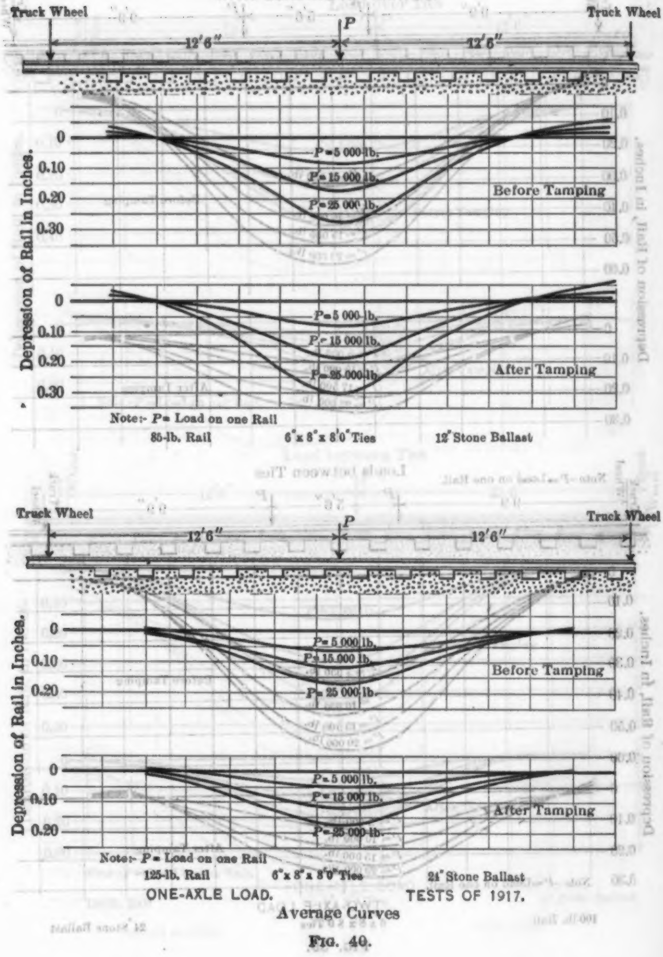


FIG. 39.

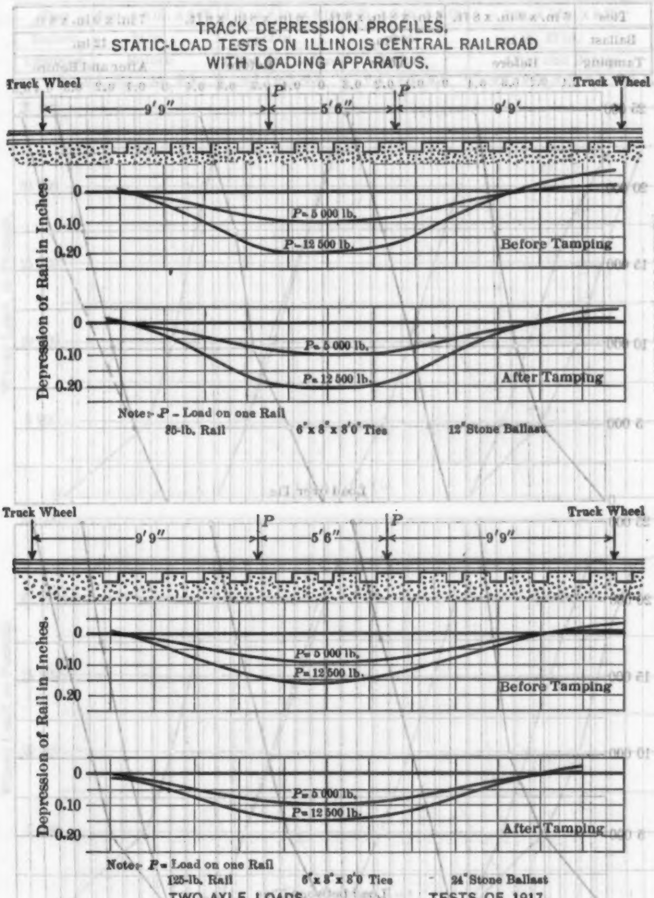
TRACK DEPRESSION PROFILES.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH LOADING APPARATUS.



ONE-AXLE LOAD. TESTS OF 1917.

Average Curves

FIG. 40.



Average Curves

FIG. 41.

LOAD-DEPRESSION DIAGRAMS.
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

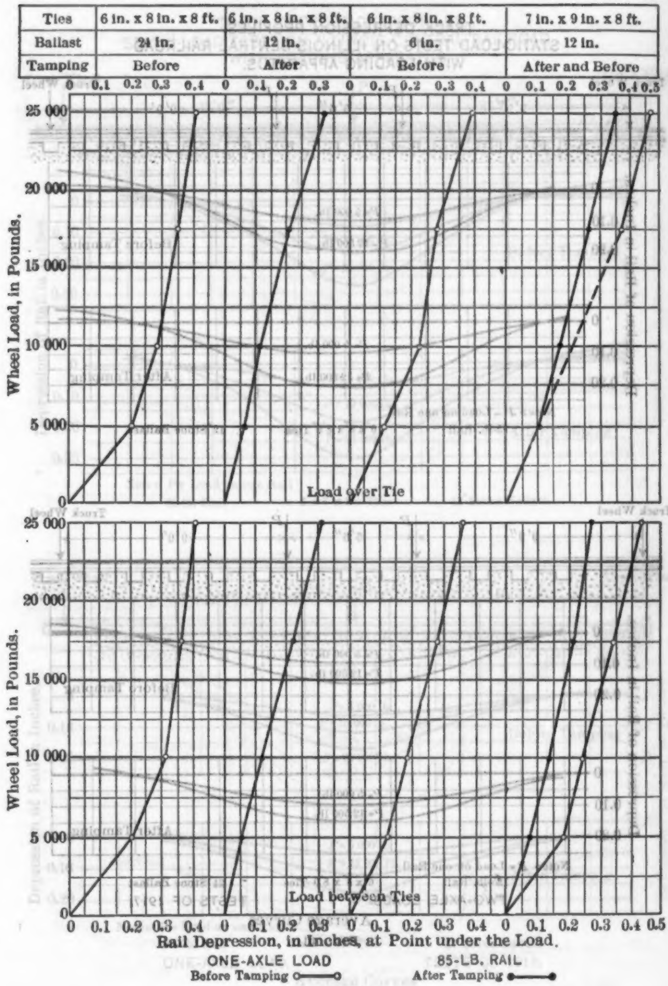


FIG. 42.

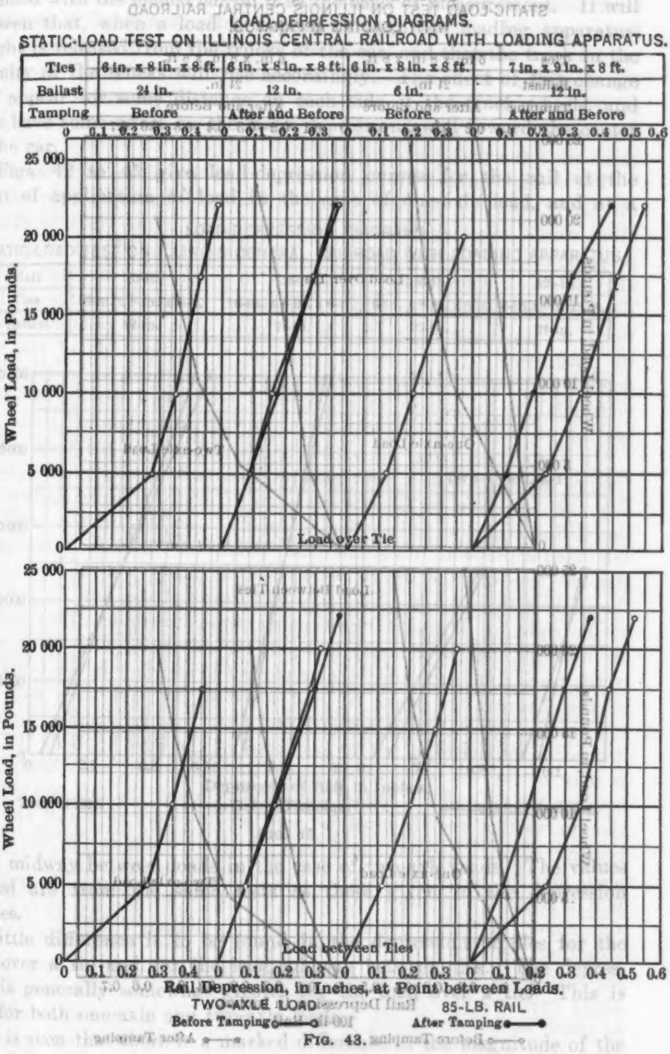

LOAD-DEPRESSION DIAGRAMS.
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.
Ties
6 in. x 8 in. x 8 ft.
6 in. x 8 in. x 8 ft.
6 in. x 8 in. x 8 ft.
7 in. x 9 in. x 8 ft.
Ballast
24 in.
12 in.
6 in.
12 in.
Tamping
Before
After and Before
Before
After and Before
Load over Tie
Load between Ties
Wheel Load, in Pounds.
Rail Depression, in Inches, at Point between Loads.
TWO-AXLE LOAD
85-LB. RAIL
Before Tamping
After Tamping

FIG. 43.

LOAD-DEPRESSION DIAGRAMS.
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD
WITH LOADING APPARATUS.

Ties	6 in. x 8 in. x 8 ft.	6 in. x 8 in. x 8 ft.
Ballast	24 in.	24 in.
Tamping	After and Before	After and Before

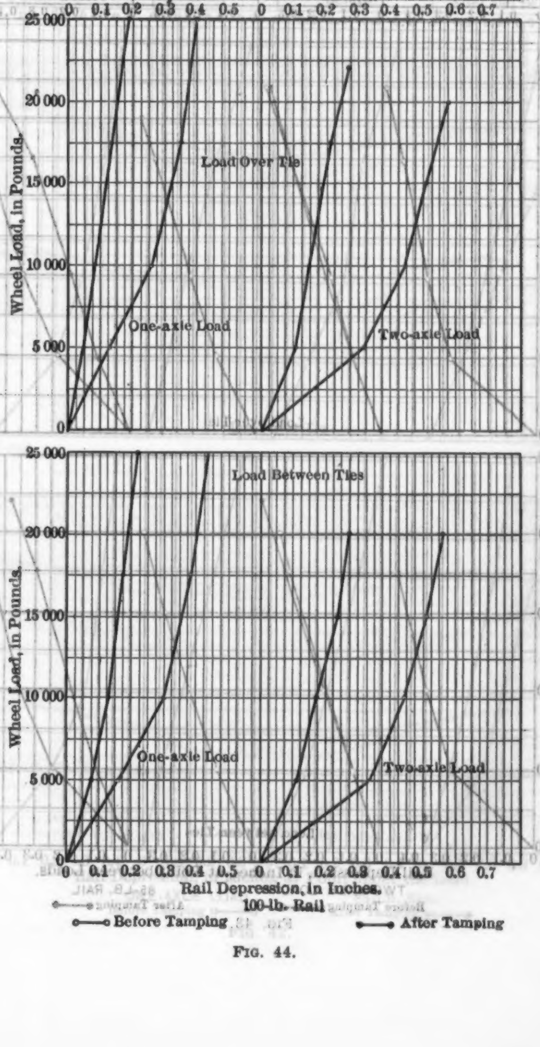


FIG. 44.

assumed with the loaded car on it as the basis of measurement. It will be seen that, when a load is applied through the loading apparatus, weight is released from the trucks of the car, and that the track in the vicinity of the trucks will rise accordingly. The effect of this change will appear for some distance on each side of the truck wheels, and may have some effect on the track depressions well toward the center of the car.

Figs. 42 to 45 give load-depression curves for the rail at the point of application of load in the case of one-axle load, and at a

LOAD-DEPRESSION DIAGRAMS.

STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

Rail	85-lb.	125-lb.	85-lb.	125-lb.
Ties	6-in. x 8-in. x 8-ft.	6-in. x 8-in. x 8-ft.	6-in. x 8-in. x 8-ft.	6-in. x 8-in. x 8-ft.
Ballast	12-in.	24-in.	12-in.	24-in.

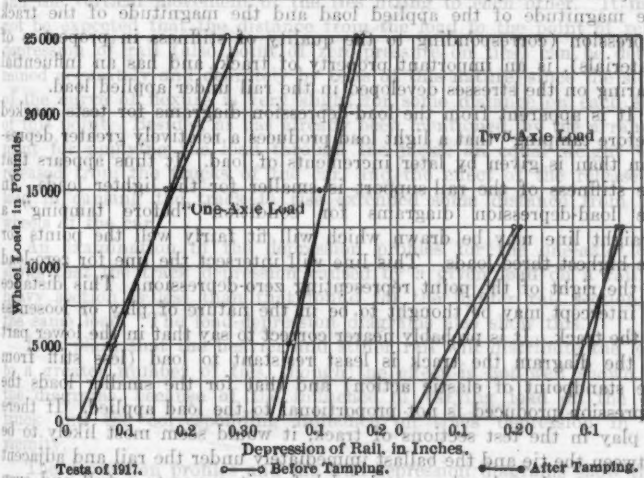


FIG. 45.

point midway between loads in the case of two-axle loads. The values plotted are from the same tests as those given in the depression profiles.

Little difference is to be found in the depression profiles for the load over a tie and for the load midway between ties. The depression is generally somewhat greater for the load over a tie. This is true for both one-axle and two-axle loads.

It is seen that there is a marked difference in the magnitude of the track depression according to the condition of the track, freshly tamped track having a smaller depression under load than track which has

been subjected to the action of traffic for a considerable time after being surfaced. In this report the term "after tamping" is applied to track on which trains had been run for, say, from 1 to 2 weeks after the track had been tamped. The term "before tamping" is applied to track which had been subjected to traffic of passenger trains for, say, from 2 to 6 months. It should be stated, however, that in all these tests the track was in excellent condition.

For freshly tamped track, a straight line drawn through the origin fits fairly well the several points in the load-depression curves. In other words, the magnitude of the depression of the track is directly proportional to the load applied. This property of direct proportionality in track depressions corresponds to a constant modulus of elasticity of rail-support as will be discussed under "Modulus of Elasticity of Rail-support." It will be found that the relation between the magnitude of the applied load and the magnitude of the track depression (corresponding to the quality of stiffness in properties of materials), is an important property of track, and has an influential bearing on the stresses developed in the rail under applied load.

It is apparent from the load-depression diagrams for tests marked "before tamping" that a light load produces a relatively greater depression than is given by later increments of load. It thus appears that the stiffness of the rail-support is smaller for the lighter loads. In the load-depression diagrams for condition "before tamping" a straight line may be drawn which will fit fairly well the points for the highest three loads. This line will intersect the line for zero-load at the right of the point representing zero-depression. This distance or intercept may be thought to be in the nature of play or looseness in the track. It is probably nearer correct to say that in the lower part of the diagram the track is least resistant to load (less stiff from the standpoint of elastic action) and that for the smaller loads the depression produced is not proportional to the load applied. If there is play in the test sections of track, it would seem most likely to be between the tie and the ballast immediately under the rail and adjacent thereto, so that the tie must bend before it comes to a full and even bearing along its length, and part of the resistance for the lighter loads may be that of the flexural resistance of the tie. It will be noted that a straight line drawn through the upper points of a load-depression curve for "before tamping" is approximately parallel to the corresponding line for freshly tamped track at the same location. The distance from the point of zero-depression to the point where the line above referred to crosses the line of zero-load has been found to be as much as 0.20 in. in the track tested, though it more generally runs from 0.03 to 0.10 in. in track not freshly tamped. How large this value may become in poorly conditioned track is an interesting question. The effect which the variation in stiffness of track found at

the several points of the load-depression curve has on the stresses developed in the rail, on the division of pressure among the ties, and on the distribution of pressure over ballast and roadway is a matter for future consideration, as is also the effect produced by changes in the magnitude of the load itself.

The same variation from direct proportionality between track depression and load (constant modulus of elasticity of rail-support) is also shown in the depression profiles for track before tamping, in that the magnitudes of the depression for the smaller loads are greater accordingly than for the larger loads. The effect is noticeable along the rail at each side of the load.

The track depression profiles for one-axle load are of interest in showing the extent of the influence of the rail in carrying load to the near-by ties and in depressing them, the curve of flexure of the rail and the vertical movement of the ties fitting to each other. It may not be expected that the distance from the load to the point of zero-depression of rail (also point of zero-pressure on ties) can be determined accurately and definitely by tests of this nature, since the slope of the curve of flexure is very slight for some distance on each side of this point. Besides, as has already been mentioned, by the arrangement for applying the load through the jacks, an equal weight was released from the trucks of the car, and the effect of the release of load in causing the track to rise extended some distance from the wheels of the car.

An examination of the depression profiles for the one-axle load shows that the track depression caused by a given load on a rail of heavy section is less than that found for the lighter rail section at the same load. The load-depression diagrams also show this.

It is plain that the heavier and stiffer rail will distribute the load to a greater number of ties, though the effect is not well shown by the diagrams, because of the influence of varying load on the car truck and the corresponding influence on track depression in its vicinity.

The depression profiles and the load-depression diagrams show that the maximum track depression for the two-axle load is greater than for a one-axle load with the same wheel load. For the wheel spacing used, the effect of one wheel load in contributing to tie pressures along the rail extends beyond the other wheel load. In Fig. 46 typical track depression profiles (shown by light lines) from one-axle load tests are plotted in duplicate as for axle loads 66 in. apart. These have been combined by summation into composite profiles as for two-axle loads with this wheel spacing of 66 in. (shown as dotted lines). The heavy full lines are depression profiles found from the two-axle load tests. It is seen that there is close agreement between the composite curve and the profile of the two-axle load test. Comparing the diagram for

85-lb. rail with that of a single truck, in Fig. 9, which is a composite diagram for two one-axle loads obtained from analysis, it is seen that there is fair agreement with the results obtained in the tests.

Attention is called to the fact that in the tests with two-axle loads (85-lb. rail) there is no reversal of curvature visible between the load points. For a less stiff rail, or for a greater wheel spacing than 5 ft. 6 in., reversal of curvature may be obtained. It will be noted that the

TRACK DEPRESSION PROFILES FROM TWO-AXLE LOAD TESTS, AND COMPOSITE PROFILES FROM ONE-AXLE LOAD TESTS.

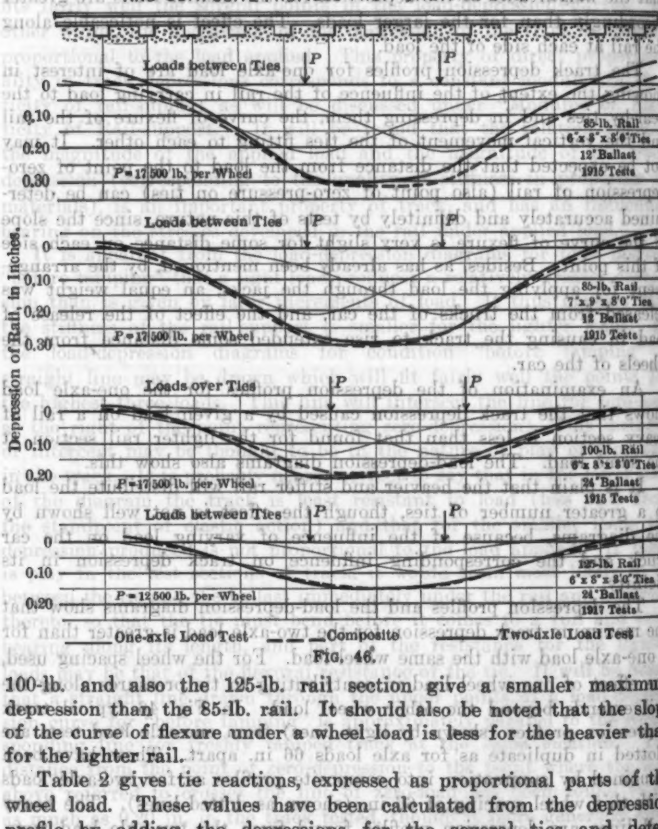
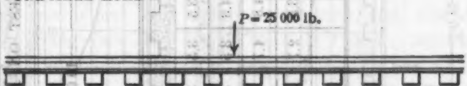
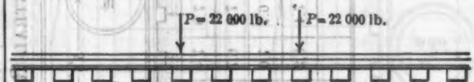
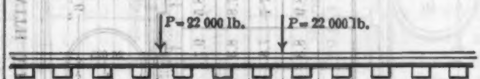


FIG. 46.

100-lb. and also the 125-lb. rail section give a smaller maximum depression than the 85-lb. rail. It should also be noted that the slope of the curve of flexure under a wheel load is less for the heavier than for the lighter rail.

Table 2 gives tie reactions, expressed as proportional parts of the wheel load. These values have been calculated from the depression profile by adding the depressions for the several ties and determining the proportion which the depression at each tie bears to their sum. The table is helpful in giving an idea of the

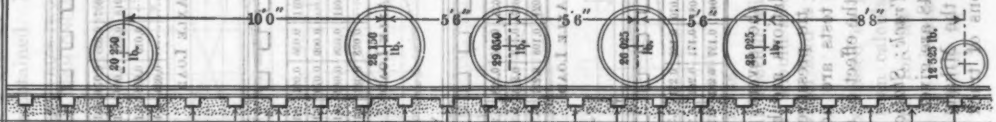
TABLE 2.—Tie Reactions, in Terms of Wheel Load, *P*. Static-Load on Illinois Central Railroad with Loading Apparatus.

Weight of Rail, in Pounds per Yard.	Size of Ties, in Inches.	Depth of Ballast in Inches.												
			One-Axle Load 											
85	6 x 8	12	......	0.007	0.035	0.073	0.119	0.167	0.196	0.167	0.119	0.073	0.035	0.007
85	7 x 9	12	0.007	0.018	0.037	0.069	0.112	0.164	0.186	0.164	0.112	0.069	0.037	0.018
100	6 x 8	24	......	......	0.015	0.052	0.115	0.193	0.248	0.193	0.115	0.052	0.015	......
125	6 x 8	24	......	......	......	0.038	0.116	0.217	0.256	0.217	0.116	0.038	......	......
			One-Axle Load 											
85	6 x 8	12	......	0.011	0.036	0.091	0.155	0.215	0.215	0.155	0.091	0.036	0.011	......
85	7 x 9	12	......	......	0.009	0.079	0.166	0.245	0.245	0.166	0.079	0.009	......	......
100	6 x 8	24	......	0.011	0.038	0.082	0.152	0.216	0.216	0.152	0.082	0.038	0.011	......
125	6 x 8	24	......	......	0.028	0.087	0.165	0.220	0.220	0.165	0.087	0.028	......	......
			Two-Axle Load 											
85	6 x 8	12	......	0.035	0.126	0.228	0.293	0.317	0.317	0.293	0.228	0.126	0.035	......
85	7 x 9	12	......	0.037	0.113	0.215	0.306	0.328	0.328	0.306	0.215	0.113	0.037	......
100	6 x 8	24	......	0.024	0.108	0.227	0.311	0.330	0.330	0.311	0.227	0.108	0.024	......
			Two-Axle Load 											
85	6 x 8	12	......	0.070	0.162	0.271	0.332	0.332	0.332	0.271	0.162	0.070	......	......
85	7 x 9	12	0.012	0.081	0.171	0.262	0.314	0.320	0.314	0.262	0.171	0.081	0.012	......
100	6 x 8	24	......	0.043	0.157	0.281	0.342	0.352	0.342	0.281	0.157	0.043	......	......

loads which come on the several ties for both one-axle and two-axle loads. The calculations are based on tests on freshly tamped track, and assume that tie pressures are proportional to depressions. The conditions of the tests are not such as to permit conclusions to be drawn relative to the effect of stiffness of rail on the distribution of load to ties.

38.—Depression of Track; Static-Load Test with Mikado Locomotive.—In Figs. 47 and 48 are given track depression profiles for static loading obtained with the Mikado locomotive on 85-lb. and 100-lb. rails on the test sections of track on the Illinois Central Railroad

TABLE 3.—TIE REACTIONS, IN THOUSANDS OF POUNDS.
STATIC-LOAD TEST ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.



Weight of Rail, in Pounds per Yard	Size of Ties, in Inches	Kind of Ballast, and depth, in Inches																							
85	6 x 8	24 Stone	3.1	4.0	4.9	4.4	3.5	3.5	3.8	4.2	6.6	7.9	7.4	8.2	8.5	8.5	9.7	9.3	9.1	9.3	7.6	4.9	3.7	8.3	3.5
85	7 x 9	12 Stone	3.5	4.5	5.7	6.0	4.3	3.5	3.4	4.5	6.4	7.0	6.7	7.1	7.8	7.8	8.3	8.8	8.4	8.3	7.5	5.6	6.5	6.0	6.5
85	6 x 8	12 Stone	3.1	4.5	5.3	5.2	4.8	4.7	5.1	5.8	6.2	6.7	7.0	7.5	7.7	7.7	7.8	7.8	7.7	7.8	7.0	5.6	4.1	3.8	4.0
85	6 x 8	6 Stone	3.9	5.2	6.0	5.2	3.5	2.7	2.6	4.0	6.5	7.4	8.2	9.6	9.6	8.9	8.8	8.5	7.5	7.8	7.1	5.1	3.9	3.4	3.3

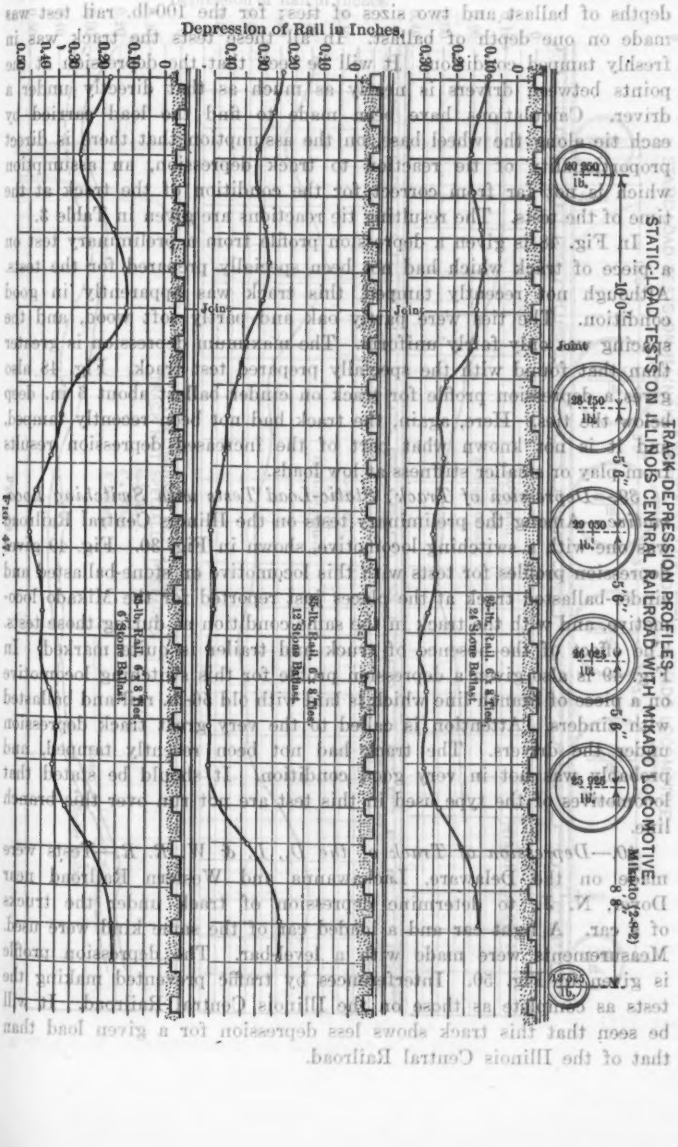

TRACK DEPRESSION PROFILES.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.
Depression of Rail in Inches.
0
0.10
0.20
0.30
0.40
0.50
Joint
Joint
Joint
30 250 lb.
28 150 lb.
29 050 lb.
26 025 lb.
25 925 lb.
12 525 lb.
10' 0"
5' 6"
5' 6"
5' 6"
8' 8"
Mikado (2-8-2)
85-lb. Rail. 6" x 8" Ties.
6" Stone Ballast.
85-lb. Rail. 6" x 8" Ties.
12" Stone Ballast.
85-lb. Rail. 6" x 8" Ties.
24" Stone Ballast.

FIG. 47.

north of Champaign. For the 85-lb. rail tests were made on three depths of ballast and two sizes of ties; for the 100-lb. rail test was made on one depth of ballast. In all these tests the track was in freshly tamped condition. It will be seen that the depression at the points between drivers is nearly as much as that directly under a driver. Calculations have been made to find the load carried by each tie along the wheel base, on the assumption that there is direct proportionality of tie reaction to track depression, an assumption which is not far from correct for the condition of the track at the time of the tests. The resulting tie reactions are given in Table 3.

In Fig. 48 is given a depression profile from a preliminary test on a piece of track which had not been specially prepared for the tests. Although not recently tamped, this track was apparently in good condition. The ties were partly oak and partly soft wood, and the spacing was only fairly uniform. The maximum depression is greater than that found with the specially prepared test track. Fig. 48 also gives a depression profile for track on cinder ballast about 6 in. deep below the ties. Here, again, the track had not been recently tamped, and it is not known what part of the increased depression results from play or smaller stiffness at low loads.

39.—Depression of Track; Static-Load Tests with Switching Locomotive.—Among the preliminary tests on the Illinois Central Railroad was one with a switching locomotive, shown in Fig. 30. Fig. 49 gives depression profiles for tests with this locomotive on stone-ballasted and cinder-ballasted track at the places just reported for the Mikado locomotive, and with the track in the same condition as during those tests. The effect of the absence of truck and trailer is quite marked. In Fig. 49 is also given a depression profile for this switching locomotive on a piece of branch line which is laid with old 56-lb. rail and ballasted with cinders. Attention is called to the very great track depression under the drivers. The track had not been recently tamped, and probably was not in very good condition. It should be stated that locomotives of the type used in this test are not run over this branch line.

40.—Depression of Track of the D., L. & W. R. R.—Tests were made on the Delaware, Lackawanna and Western Railroad near Dover, N. J., to determine depression of track under the trucks of a car. A light car and a loaded car of the same kind were used. Measurements were made with a level-bar. The depression profile is given in Fig. 50. Interferences by traffic prevented making the tests as complete as those on the Illinois Central Railroad. It will be seen that this track shows less depression for a given load than that of the Illinois Central Railroad.

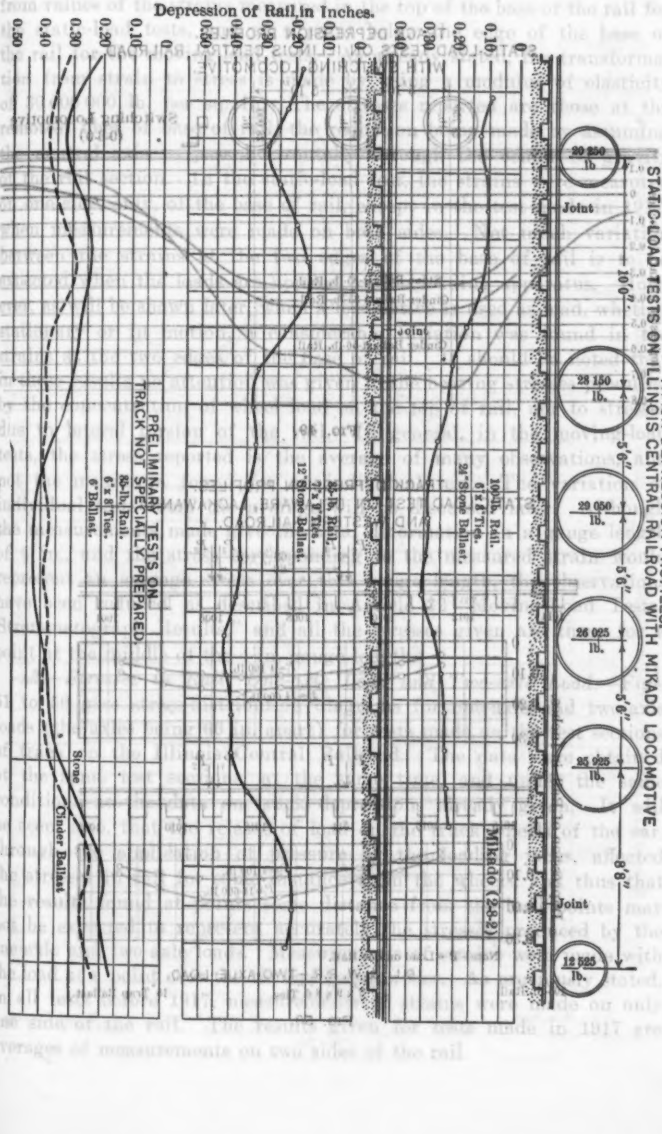

TRACK DEPRESSION PROFILES.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE
20 250 lb.
28 150 lb.
29 050 lb.
26 025 lb.
25 925 lb.
12 525 lb.
10'0"
5'6"
5'6"
5'6"
8'8"
Joint
Joint
Depression of Rail, in Inches.
0
0.10
0.20
0.30
0.40
0.50
100-lb. Rail.
6"x 8" Ties.
24" Stone Ballast.
Mikado (2-8-2)
85-lb. Rail.
7"x 9" Ties.
12" Stone Ballast.
PRELIMINARY TESTS ON
TRACK NOT SPECIALLY PREPARED.
85-lb. Rail.
6"x 8" Ties.
6" Ballast.
Stone
Cinder Ballast

Fig. 48.

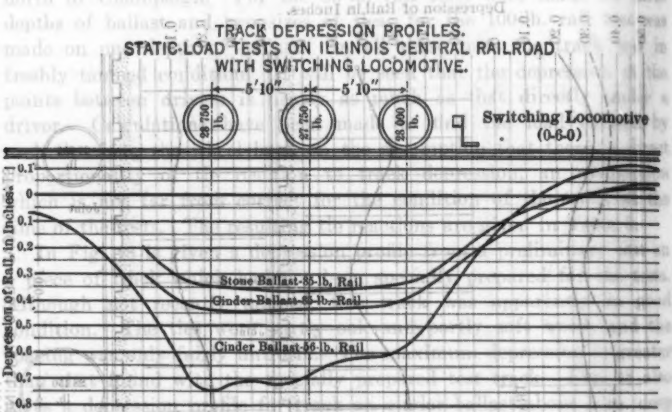

TRACK DEPRESSION PROFILES.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH SWITCHING LOCOMOTIVE.
5'10"
5'10"
28 750 lb.
27 750 lb.
28 000 lb.
Switching Locomotive (0-6-0)
Depression of Rail, in Inches.
Stone Ballast-85-lb. Rail
Cinder Ballast-85-lb. Rail
Cinder Ballast-56-lb. Rail

FIG. 49.

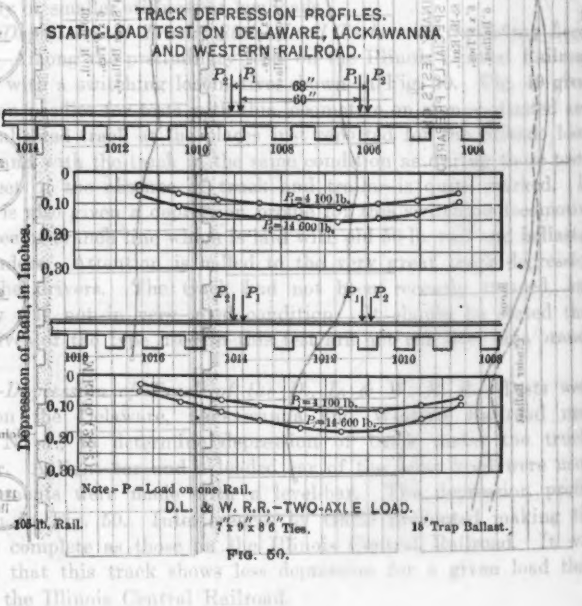

TRACK DEPRESSION PROFILES.
STATIC-LOAD TEST ON DELAWARE, LACKAWANNA AND WESTERN RAILROAD.
68"
60"
Depression of Rail, in Inches.
P_1=4 100 lb.
P_2=14 600 lb.
Note:- P = Load on one Rail.
D.L. & W. R.R.-TWO-AXLE LOAD.
105-lb. Rail.
7" x 9" x 8' 6" Ties.
18" Trap Ballast.

FIG. 50.

41.—Stresses in Rail.—The stresses reported here are calculated from values of the strains measured in the top of the base of the rail for the static-load tests, and at a point below the edge of the base of the rail for the moving-load tests. As already stated, the transformation from strain to stress is made by using a modulus of elasticity of 30 000 000 lb. per sq. in. The stresses reported are those at the remotest fiber of base of rail, the reduction being made by assuming the neutral axis to pass horizontally through the center of gravity of the rail section. In the static-load test, the strains were measured on one side, only, of the base of rail, except in the test made in 1917, when measurements were made on both sides. Not much variation between the strains at the two edges of the base of rail is to be expected when the loads are applied by the loading apparatus. However, as will be shown later, when a locomotive is used as load, whether stationary or in motion, a considerable variation was found in the strains at the two edges of the base of rail. It should be noted that in these results no attention was given to the bearing stresses produced by the concentration of wheel load on the top of rail, nor to stresses due to lateral torsion of the rail. In general, in the moving-load tests, the stress reported is the average of many observations, and not the maximum found for an individual test. The variation of individual tests from the average will be discussed later. Although the measurements made give the total deformation in a gauge length of 4 in., and the stress corresponding to the measured strain would represent an average stress over this gauge length, the observations have been reduced, as described in Article 32 "Moving-Load Tests; Stremmatograph Results," and all the stresses given are those for a point at the middle of the 4-in. gauge length.

42.—Stresses in Rail; One-axle Load and Two-axle Load.—Figs. 51 to 59 give stress-distribution diagrams for one-axle and two-axle loads (the axles being 66 in. apart) for tests made on the test sections of track on the Illinois Central Railroad. The data were obtained at the same test sections, at the same time, and under the same conditions as the data on track depressions already given. It will be seen, also, that the release of load on the truck wheels of the car, through the application of pressure by the loading jacks, affected the stresses in rail for some distance from the wheels, and thus that the results found at points some distance from the load points may not be expected to represent accurately the stresses produced by the one-axle and two-axle loads. Measurements of strains were made with the load at a point over the tie and between ties. As previously stated, in all tests before 1917, measurements of strains were made on only one side of the rail. The results given for tests made in 1917 are averages of measurements on two sides of the rail.

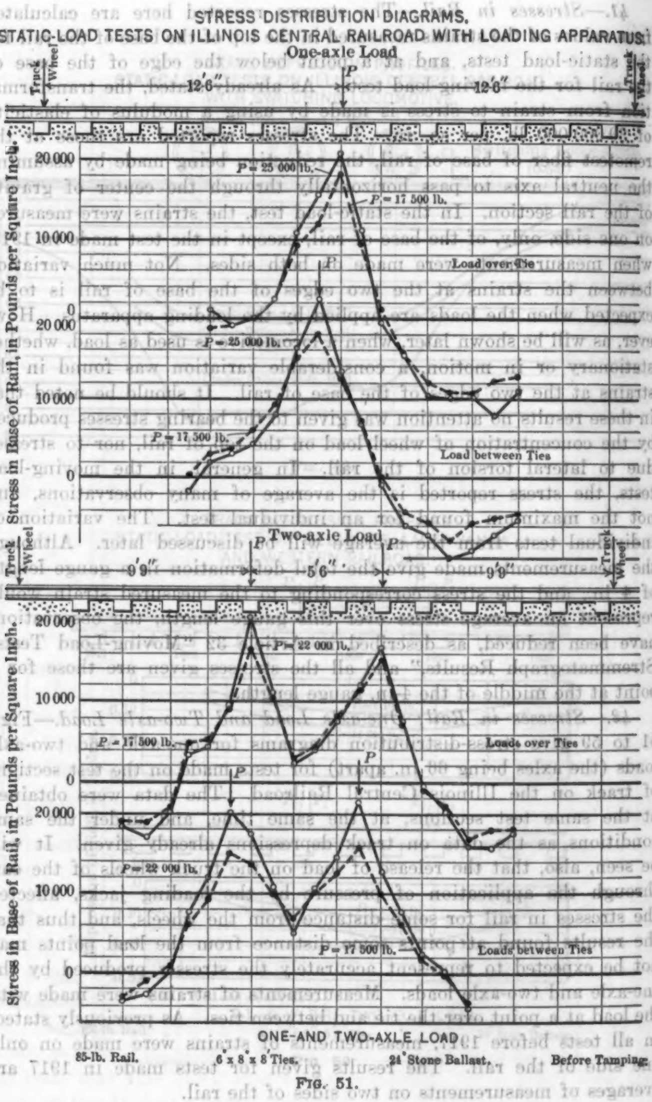
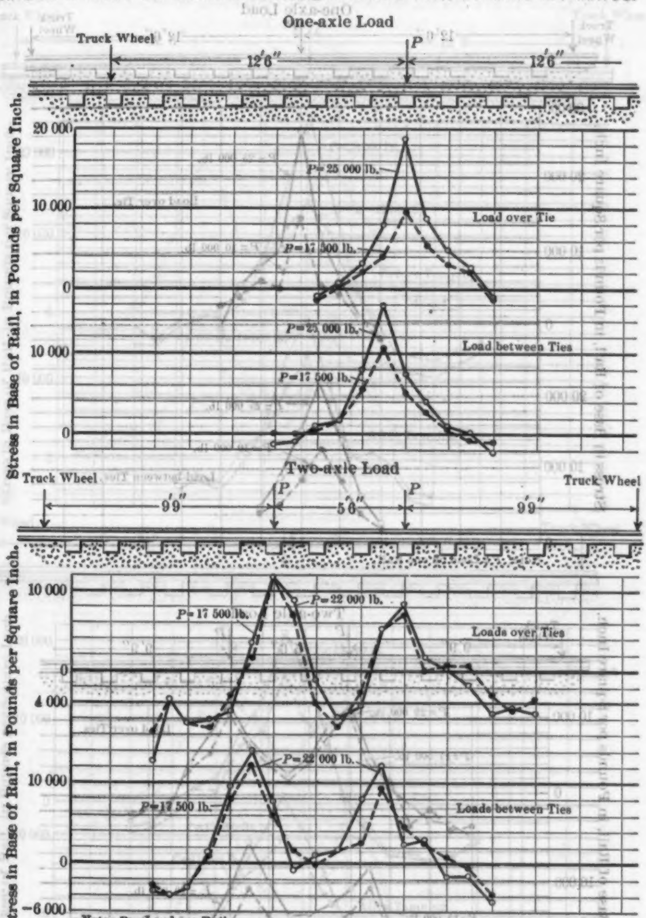

STRESS DISTRIBUTION DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS
One-axle Load
Truck Wheel
12'6"
P
12'6"
Truck Wheel
Stress in Base of Rail, in Pounds per Square Inch.
20 000
10 000
0
P = 25 000 lb.
P = 17 500 lb.
Load over Tie
P = 25 000 lb.
P = 17 500 lb.
Load between Ties
Two-axle Load
9'9"
5'6"
9'9"
P = 22 000 lb.
P = 17 500 lb.
Loads over Ties
P = 22 000 lb.
P = 17 500 lb.
Loads between Ties
ONE-AND TWO-AXLE LOAD
85-lb. Rail.
6" x 8" x 8' Ties.
24" Stone Ballast.
Before Tamping.

FIG. 51.

STRESS DISTRIBUTION DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.



ONE-AND TWO-AXLE LOAD

85-lb. Rail. 6"x 8"x 8' Ties. 12" Stone Ballast. After Tamping.

FIG. 52.

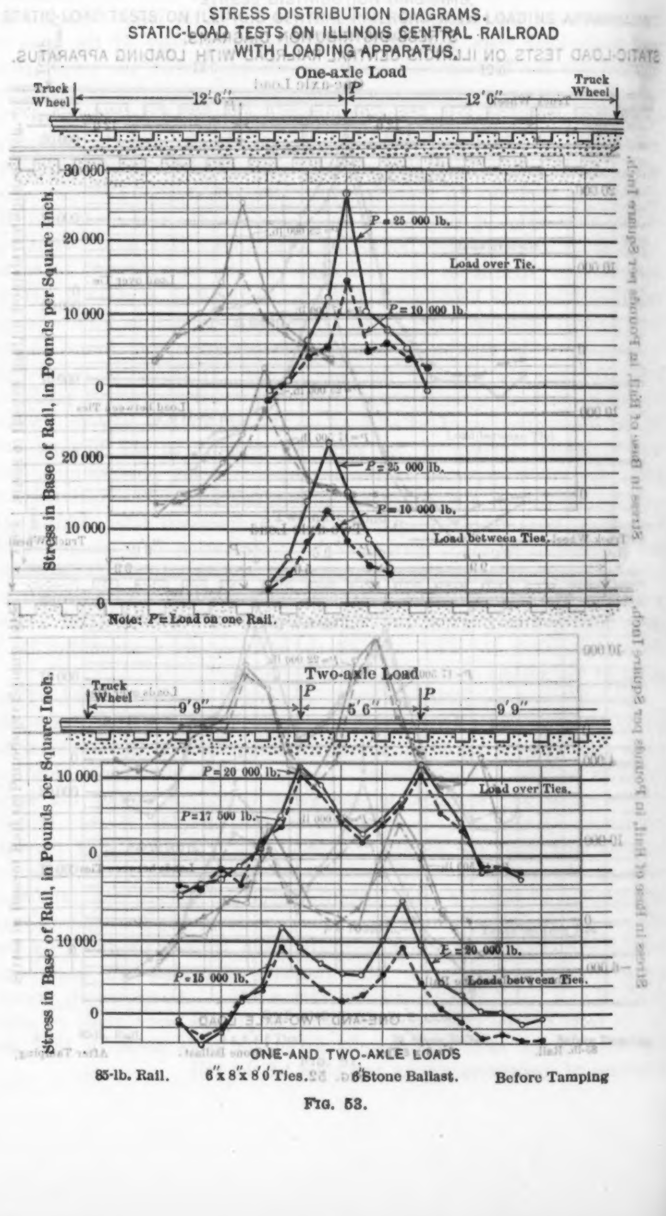

STRESS DISTRIBUTION DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH LOADING APPARATUS.
One-axle Load
Truck Wheel
12′ 6″
P
12′ 6″
Truck Wheel
Stress in Base of Rail, in Pounds per Square Inch.
30 000
20 000
10 000
0
P = 25 000 lb.
Load over Tie.
P = 10 000 lb.
20 000
10 000
0
P = 25 000 lb.
P = 10 000 lb.
Load between Ties.
Note: P = Load on one Rail.
Two-axle Load
Truck Wheel
9′ 9″
P
5′ 6″
P
9′ 9″
Stress in Base of Rail, in Pounds per Square Inch.
10 000
0
P = 20 000 lb.
P = 17 500 lb.
Load over Ties.
10 000
0
P = 20 000 lb.
P = 15 000 lb.
Loads between Ties.
ONE-AND TWO-AXLE LOADS
85-lb. Rail.
6″ x 8″ x 8′ 0 Ties.
6″ Stone Ballast.
Before Tamping

FIG. 53.

STRESS DISTRIBUTION DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

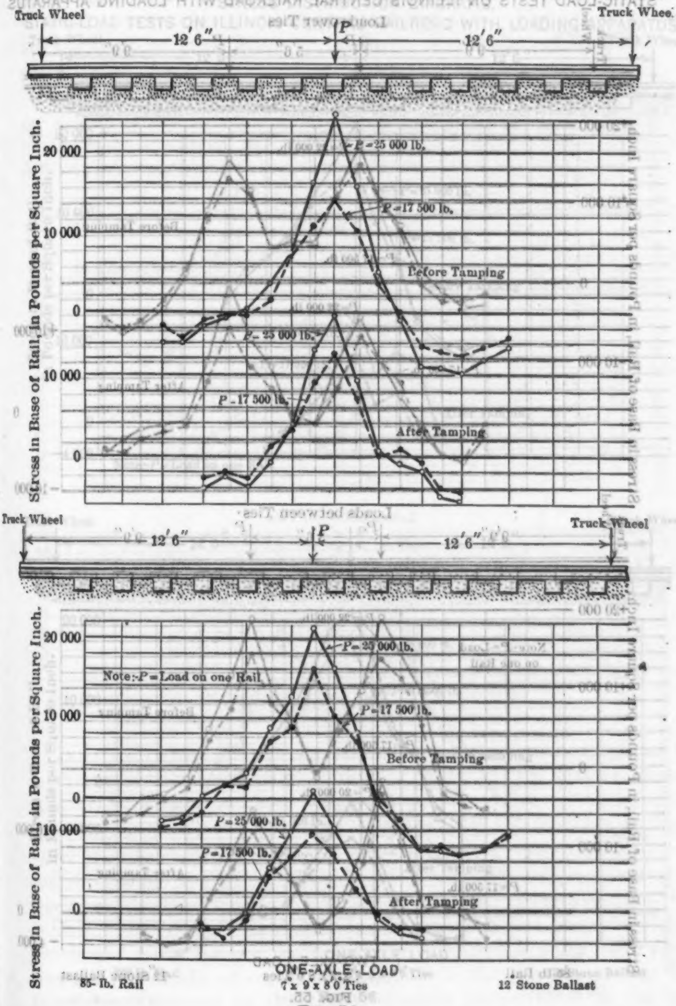


FIG. 54.

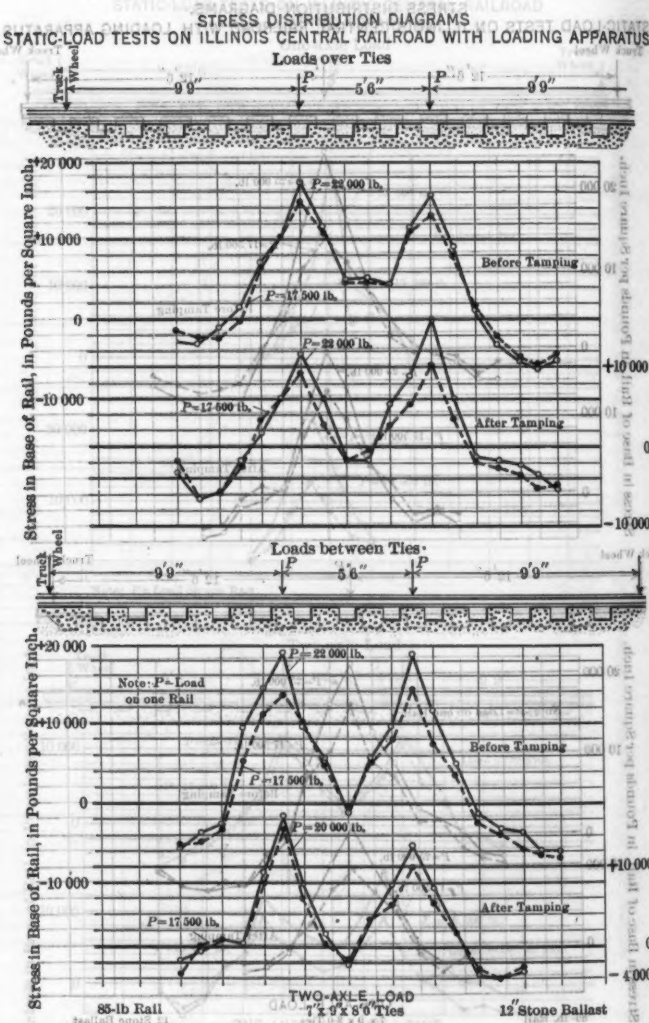

STRESS DISTRIBUTION DIAGRAMS
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS
Loads over Ties
Truck Wheel
P
P
9′9″
5′6″
9′9″
Stress in Base of Rail, in Pounds per Square Inch.
+20 000
+10 000
0
-10 000
+10 000
0
-10 000
P=22 000 lb.
P=17 500 lb.
Before Tamping
P=22 000 lb.
P=17 500 lb.
After Tamping
Loads between Ties
Truck Wheel
P
P
9′9″
5′6″
9′9″
Stress in Base of Rail, in Pounds per Square Inch.
+20 000
+10 000
0
-10 000
+10 000
0
-4 000
Note: P=Load on one Rail
P=22 000 lb.
P=17 500 lb.
Before Tamping
P=20 000 lb.
P=17 500 lb.
After Tamping
TWO-AXLE LOAD
85-lb Rail
7″x 9″x 8′6″ Ties
12″ Stone Ballast

FIG. 55.

STRESS DISTRIBUTION DIAGRAMS.

STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

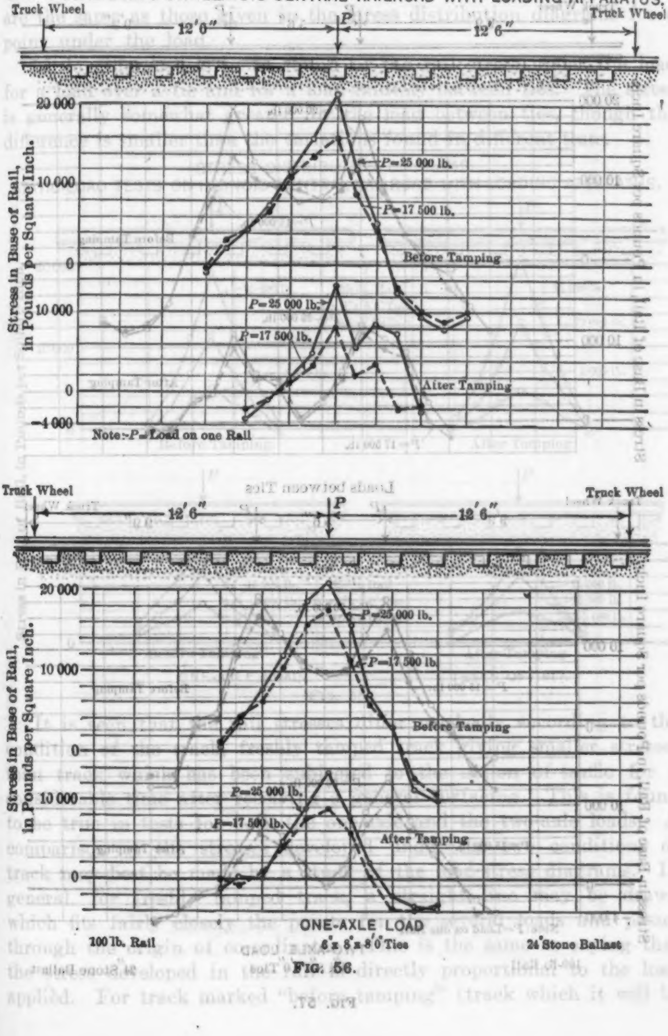


FIG. 56.

STRESS DISTRIBUTION DIAGRAMS
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

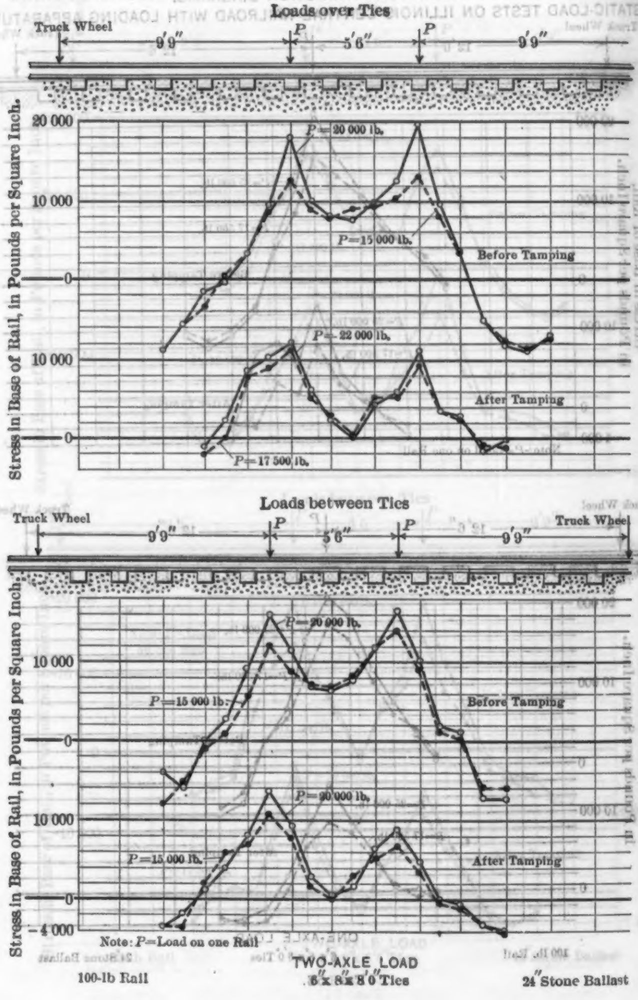


FIG. 57.

Figs. 60 to 63 give load-stress diagrams for gauge lines at the point of application of load in the case of one-axle load and at the points of load in the case of two-axle loads, the average of the stresses at the two points being taken in the latter case. The values plotted are the same as those given in the stress distribution diagrams for a point under the load.

Little difference is to be found in the rail stress under the load for a load over a tie and for a load midway between ties. The stress is generally somewhat greater for the load between ties, though the difference is smaller than the variations found in different tests.

STRESS DISTRIBUTION DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

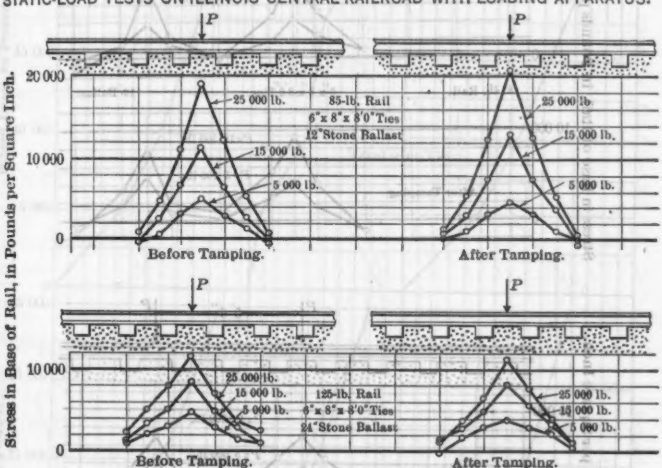
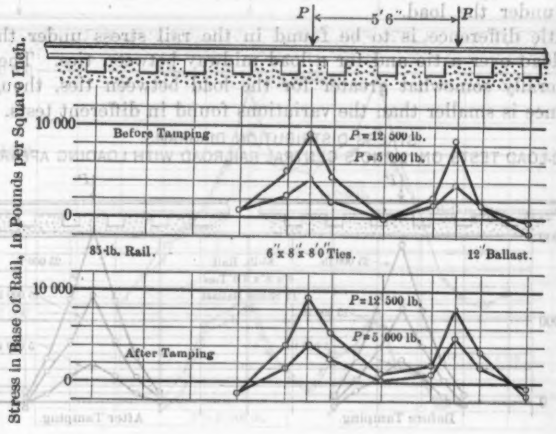


FIG. 58.

It is seen that the rail stresses differ markedly according to the condition of the track, freshly tamped track giving smaller stresses than track which has been subjected to the action of traffic for a considerable time after receiving a general surfacing. This is found to be true in tests for both the one-axle and the two-axle loads. A comparison of the stresses developed under the two conditions of track may best be made by a study of the load-stress diagrams. In general, for freshly tamped track, a straight line may be drawn which fits fairly closely the points for the several loads and passes through the origin of co-ordinates. This is the same as saying that the stress developed in the rail is directly proportional to the load applied. For track marked "before tamping" (track which it will be

STRESS DISTRIBUTION DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH LOADING APPARATUS.



TWO-AXLE LOAD. TESTS OF 1917.
Average of Stresses on two sides of Rail.

FIG. 59.

LOAD-STRESS DIAGRAMS.

STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

Ties.	6" x 8" x 8' 0"	6" x 8" x 8' 0"	6" x 8" x 8' 0"
Ballast.	24"	12"	6"
Tamping.	Before.	After and Before.	Before.

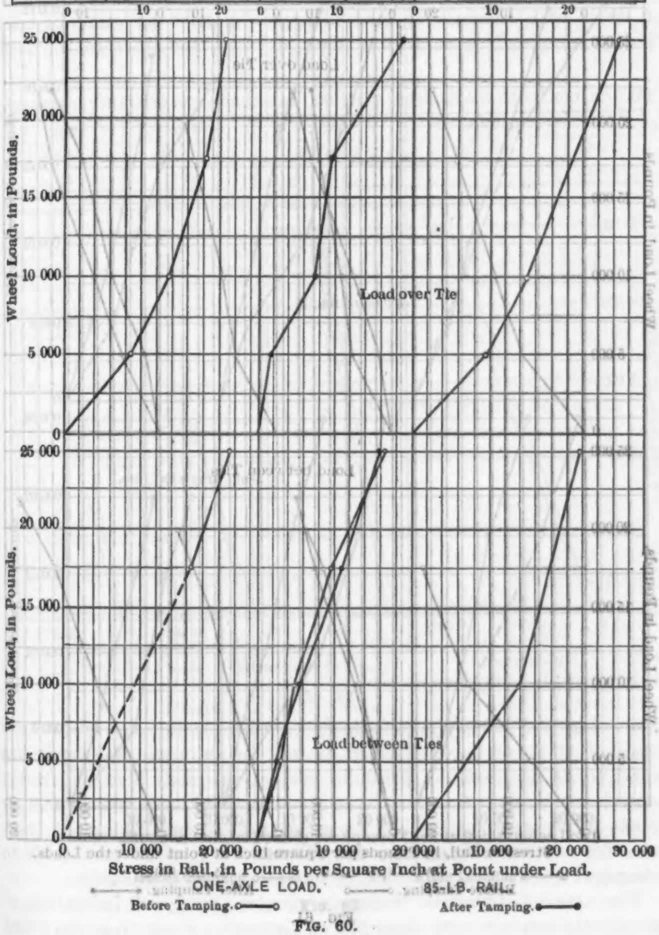
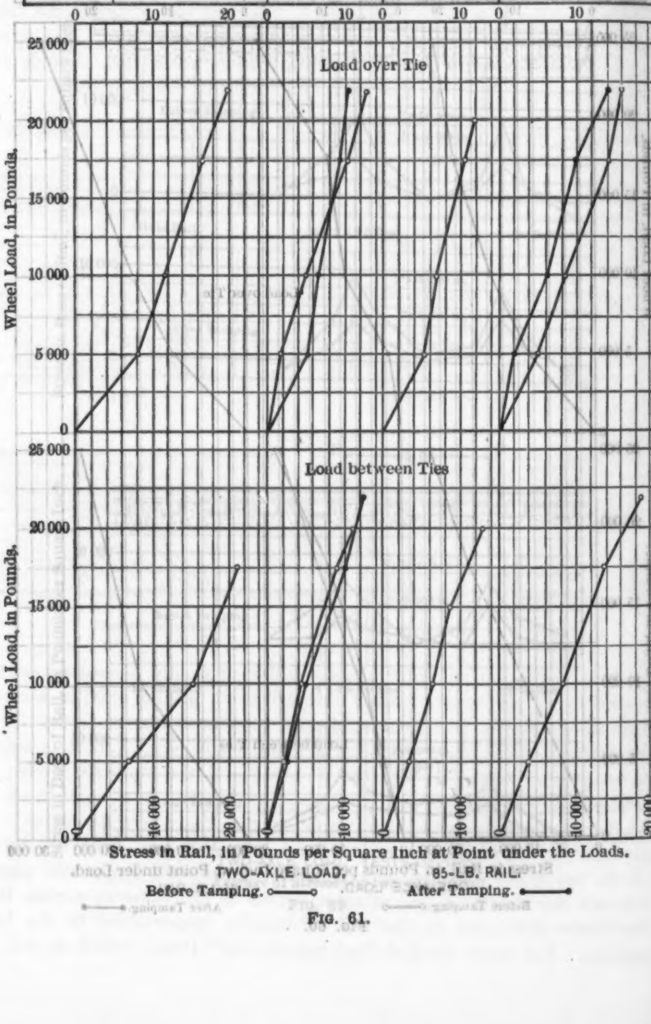


FIG. 60.

LOAD-STRESS DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

Ties.	6"x 8"x 8'0"	6"x 8"x 8'0"	6"x 8"x 8'0"	7"x 9"x 8'0"
Ballast.	24"	12"	6"	12"
Tamping.	Before.	After and Before.	Before.	After and Before.



Stress in Rail, in Pounds per Square Inch at Point under the Loads.
TWO-AXLE LOAD. 85-LB. RAIL.
Before Tamping. ○——○ After Tamping. ●——●

FIG. 61.

LOAD-STRESS DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

Ties.	6″x 8″x 8′	6″x 8″x 8′	7″x 9″x 8′
Ballast.	24″	24″	12″
Tamping.	After and Before.	After and Before.	After and Before.

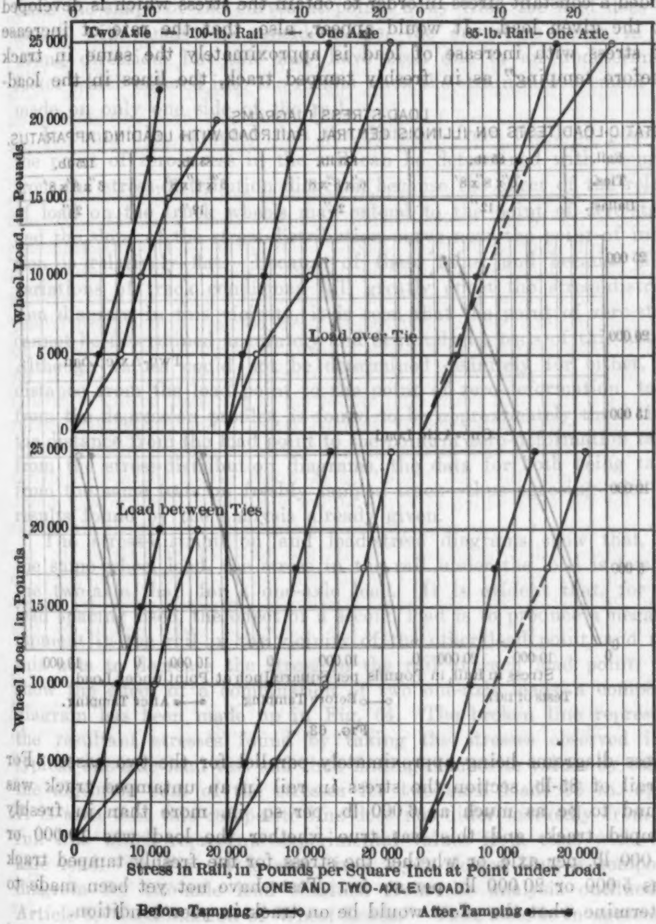


FIG. 62.

remembered has been described as track in excellent condition), a straight line may be drawn to fit fairly well the points for the several loads, but this line will pass to the right of the origin. For the track "before tamping", then, the stresses appear to increase proportionately to the load; but, to the stress found on this assumption must be added a constant stress in order to obtain the stress which is developed at the given load. It would appear, also, that the rate of increase of stress with increase of load is approximately the same in track "before tamping" as in freshly tamped track, the lines in the load-

LOAD-STRESS DIAGRAMS.

STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH LOADING APPARATUS.

Rail.	85 lb.	125 lb.	85 lb.	125 lb.
Ties.	6″x 8″x 8′	6″x 8″x 8′	6″x 8″x 8′	6″x 8″x 8′
Ballast.	12″	24″	12″	24″

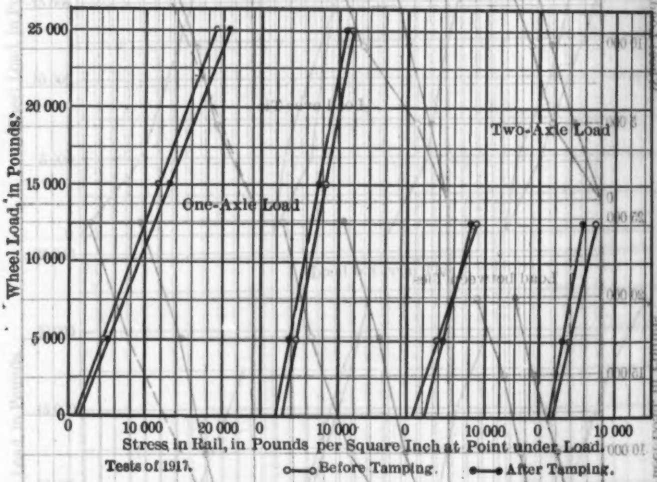


FIG. 63.

stress diagrams being approximately parallel for the two cases. For a rail of 85-lb. section the stress in rail in an untamped track was found to be as much as 6 000 lb. per sq. in. more than in freshly tamped track, and this was true whether the load was 10 000 or 50 000 lb. per axle, or whether the stress for the freshly tamped track was 5 000 or 20 000 lb. per sq. in. Tests have not yet been made to determine what the effect would be on track in poor condition.

The stress-distribution diagrams give a very good indication of the way the stresses vary along the rail at points away from the wheel

load, and also the way the moments change from positive to negative. It will be seen that the shape of the curves and the distribution of stresses along the rail are very similar to the distribution found by the analysis heretofore given, as illustrated in Fig. 9. In a few cases the shape of the curves at points away from the load does not accord with analytical considerations, the curvature being in the wrong direction, but this may have been due to an undetermined factor which entered into the results when the strain measurement was made on only one side of the rail.

It cannot be expected that the distance from the load point to the point of zero-stress in the rail can be determined with accuracy from the stress-distribution diagram, because the effect of the release of load on the truck wheels may extend to the point of zero-stress, and the slope of the stress-distribution curve near the point of inflection is relatively flat. Because of these facts, and because slight variations of track conditions will greatly affect the stress-distribution diagram in this vicinity, it is seen that the point of zero-stress cannot be determined accurately and definitely by tests of this nature. Although values could not be determined definitely for either, the distance from the load point to the point of zero-deformation, taken from the depression profiles, is found to be approximately three times the distance from the load point to the point of zero-deformation taken from the stress-distribution diagrams, the data for both being taken from the same tests on freshly tamped track—thus agreeing with the results found by the analysis already given.

The stress-distribution and load-stress diagrams show that, for the same wheel load, the stress in the rail under the load is less for the two-axle than for a one-axle load. It is evident that, for the load spacing used, the effect of a second load is to produce a negative moment in the rail in the vicinity of the other load point, and thus this acts to decrease the stress in the rail at each load point. To show the effect of a combination of two one-axle loads, a composite diagram has been made up in Fig. 64. The broken line represents the resultant stresses found by taking the stresses observed in a typical one-axle load test and then finding composite values by taking the algebraic sum of the stresses for two such loads 66 in. apart. It is seen that this composite line does not differ markedly from the full line which represents experimental values from tests with two-axle load. In Fig. 64 is also given, by a dotted line, a composite diagram for two-axle load obtained from the analysis outlined in Article 6, "Combination of Wheel Loads." This, also, is in agreement with the results obtained in the tests.

To bring out the fact that relations exist in the stresses and moments along the rail which vary with the spacing of the wheels

and the properties of the track, there are given in Fig. 65 three composite bending moment distribution diagrams for two-axle loads which were made up by using the analysis heretofore given. The upper one (*a*) is based on a value of x_1 (the distance from a single wheel load to the point of zero bending moment in the rail) equal to two-thirds of the distance between the loads; the middle one (*b*) on a value of x_1 equal to one-half, and the lower one (*c*) on a value of x_1 equal to one-third, of the distance between loads. The value which may be expected to apply to track in any case will depend on (1) the spacing of the loads, (2) the rail section used, and (3) the stiffness of the rail support.

In the tests with two-axle load on 85-lb. rail the stress-distribution diagrams for freshly tamped track generally show a negative moment at a point midway between loads. The form of the diagram corresponds to the form obtained by using in the analysis a value of x_1 (the distance from a single wheel load to the point of zero bending moment) of about 28 in. For rail of 85-lb. section, in track not freshly tamped, the moment at a point midway between loads, as shown by the diagrams, runs from zero to a positive moment of 0.25 of the moment at the load point, and the stress at the load points is greater than was found with freshly tamped track. For those tests which give a moment of zero at the midway point, it appears from the load-stress diagrams that the track was in much better surface condition than for the tests which give the higher moment at the midway point. For the highest loads used in these tests, it would appear, from the stress-distribution diagrams, as if the axis of zero-stress in the diagrams for freshly tamped track had been lowered from 5 000 to 7 000 lb. per sq. in., the stress-distribution diagrams for track not in freshly tamped condition giving about the same results as this changed diagram.

For the rail of 100-lb. section, the stress-distribution diagrams for freshly tamped track show a zero moment at a point midway between loads. This form of the diagram corresponds in the analysis to a value of the distance from a single wheel load to the point of zero bending moment, x_1, of about 33 in., one-half of the distance between points of applied load. In track not freshly tamped, the moment at the point midway between loads is found to be positive and equal to about four-tenths of the moment at the wheel load, and the stress under the wheel load is greater than for track freshly tamped. For the higher loads, the same comment may be made, the diagrams for track not freshly tamped bearing a resemblance to those for freshly tamped track with the axis of zero-stress moved downward about 6 000 lb. per sq. in.

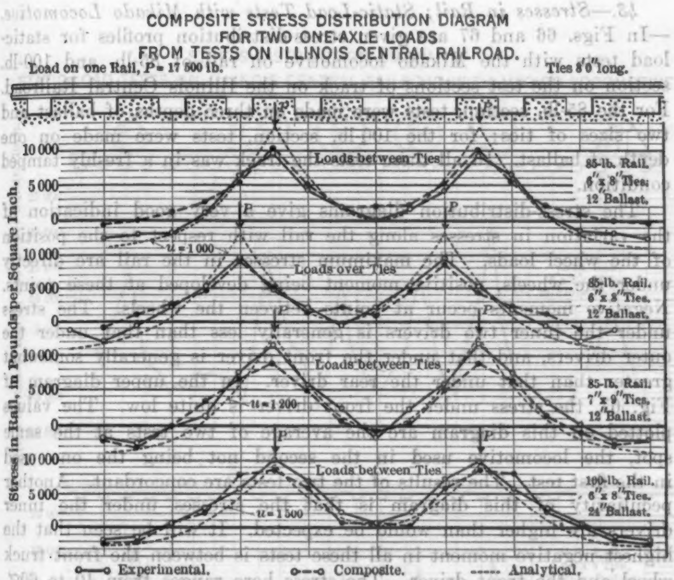

COMPOSITE STRESS DISTRIBUTION DIAGRAM
FOR TWO ONE-AXLE LOADS
FROM TESTS ON ILLINOIS CENTRAL RAILROAD.
Load on one Rail, P = 17 500 lb.
Ties 8'0" long.
Stress in Rail, in Pounds per Square Inch.
Loads between Ties
85-lb. Rail. 6" x 8" Ties. 12" Ballast.
u = 1 000
Loads over Ties
85-lb. Rail. 6" x 8" Ties. 12" Ballast.
Loads between Ties
85-lb. Rail. 7" x 9" Ties. 12" Ballast.
u = 1 200
Loads between Ties
100-lb. Rail. 6" x 8" Ties. 24" Ballast.
u = 1 500
Experimental.
Composite.
Analytical.

FIG. 64.

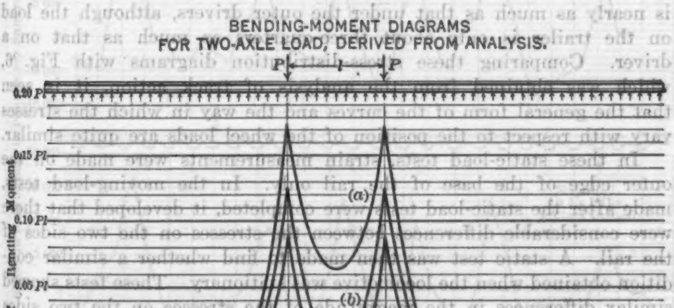

BENDING-MOMENT DIAGRAMS
FOR TWO-AXLE LOAD, DERIVED FROM ANALYSIS.
P
l
P
0.20 Pl
0.15 Pl
0.10 Pl
0.05 Pl
0
-0.05 Pl
Values of Bending Moment
(a)
(b)
(c)

FIG. 65.

43.—Stresses in Rail; Static-Load Tests with Mikado Locomotive. —In Figs. 66 and 67 are given stress-distribution profiles for static-load tests with the Mikado locomotive on rail of 85-lb. and 100-lb. section on the test sections of track on the Illinois Central Railroad. For the 85-lb. section, tests were made on three depths of ballast and two sizes of ties; for the 100-lb. section, tests were made on one depth of ballast. In all these tests the track was in a freshly tamped condition.

The stress-distribution diagrams give a very good indication of the variation in stresses along the rail with respect to the position of the wheel loads. The maximum stresses in the rail are directly under the wheels, positive moment being developed at these points. Negative moments occur at points between the wheels. The stress under the inner two drivers is generally less than that under the outer drivers, and that under the front driver is generally somewhat greater than that under the rear driver. In the upper diagram of Fig. 67, the stress under the front driver is quite low. The values plotted on this diagram are the average of two tests at the same spot, the locomotive used in the second not being the one used in the first test. The results of the two tests are concordant. Another peculiarity of this diagram is that the stresses under the inner drivers are higher than would be expected. It will be seen that the highest negative moment in all these tests is between the front truck wheel and the front driver. The stress here ranges from 40 to 60% of that developed under the front driver. The stress under the trailer is nearly as much as that under the outer drivers, although the load on the trailer is only about three-quarters as much as that on a driver. Comparing these stress-distribution diagrams with Fig. 6, which was obtained from the analysis of track action, it is seen that the general form of the curves and the way in which the stresses vary with respect to the position of the wheel loads are quite similar.

In these static-load tests, strain measurements were made on the outer edge of the base of the rail only. In the moving-load tests, made after the static-load tests were completed, it developed that there were considerable differences between the stresses on the two sides of the rail. A static test was then made to find whether a similar condition obtained when the locomotive was stationary. These tests showed similar differences in the magnitude of the stresses on the two sides of the rail. The values found for the static-load tests with the locomotive, given in Figs. 66 and 67, then, may not be expected to represent accurately the average stress in the base of the rail. The difference between the stress on the outside of the rail and the average stress on the two sides, as will be shown later, has been found to vary in different tests. However, the results of the strain-gauge measurements on only one side of the rail are of interest and value in a number

STRESS DISTRIBUTION DIAGRAMS.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.

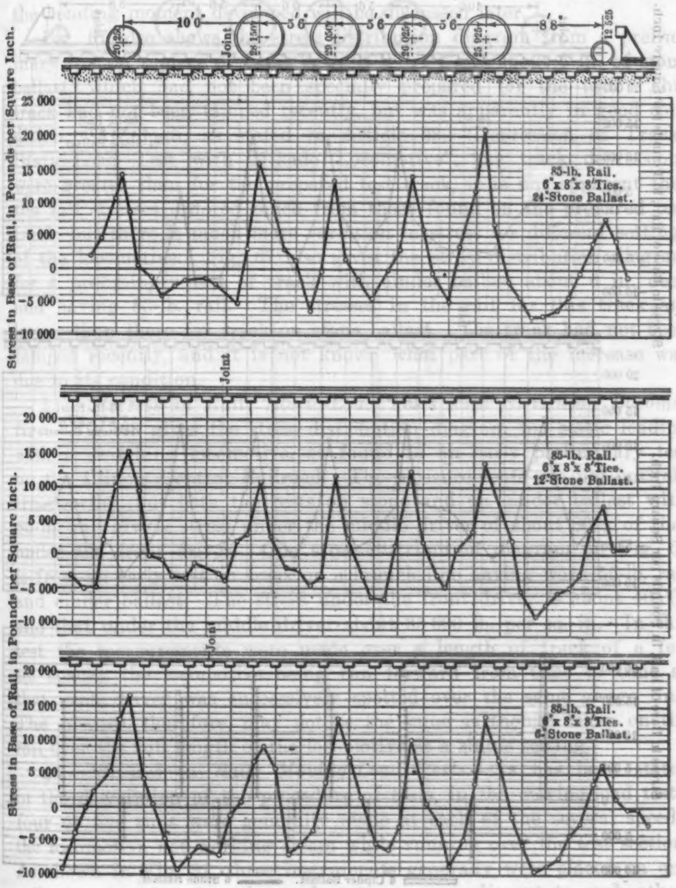
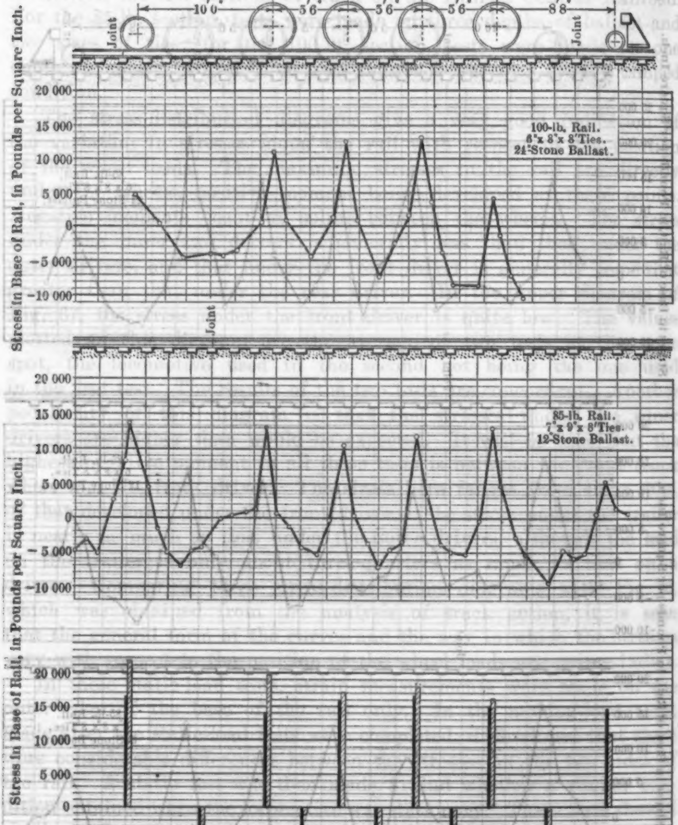


FIG. 66.

STRESS DISTRIBUTION DIAGRAMS.

STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.



PRELIMINARY TEST.

FIG. 67.

of ways, and they may be considered to show, in a general way, the distribution of stress along the rail under the given loading.

The tests on rail of 100-lb. section gave lower stresses, as might be expected, but the general distribution of stresses along the rail was much the same. The effect of the increased section of rail on the bending moment developed will be discussed later.

Fig. 67 also shows the stress-distribution diagram from a preliminary test on a piece of track with 85-lb. rail and about 6 in. of stone ballast, which had not been specially prepared for the test. This track had not been tamped recently, but was apparently in good condition. Although, as stated in Article 38, "Depression of Track; Static-Load Test with Mikado Locomotive", the track depressions were greater than for the prepared test track, it is not apparent that the rail stresses differed much from those found on the prepared test track, but there is much more variation in stress for different settings of the locomotive. Fig. 67 also gives the stress-distribution diagram for a preliminary test of track with cinder ballast about 6 in. deep and having 85-lb. rail. The stresses in the rail for this track are larger than those for track on stone ballast. The track had not been tamped recently, and it is not known what part of the increase was due to its condition.

44.—Stresses in Rail; Static-Load Tests with Switching Locomotive.—Fig. 68 gives the stress-distribution diagram for static loading with a switching locomotive, as found in an early preliminary test on the Illinois Central Railroad. The measurements are not entirely trustworthy, but both the tests on stone and on cinder ballast with 85-lb. rail give a stress under the middle driver of about 80% of that under the front driver. One stress-distribution diagram on Fig. 68 is from an early test on track on a branch line having worn 56-lb. rail and cinder ballast. The stress under the front driver is about 40 000 and that under the middle driver about 35 000 lb. per sq. in. In this test the measurements were made over a length of track of a few tie spaces, the locomotive being run forward from time to time so that each driver was successively spotted over the same gauge line. The stresses, therefore, may not be the same as though taken on the rail over the full length of the locomotive at a single setting.

45.—Stresses in Rail; Moving-Load Tests.—As has been stated in the description of the procedure of tests, in the moving-load tests, four or five runs were generally made at each of the several speeds, the locomotive not working steam while running over the test section. As shown in Fig. 31, three instruments generally were placed on one rail at points between ties and at tie spaces adjacent to each other. The fourth instrument was placed on the other rail opposite the middle one of the three instruments. As the position of the locomotive driver was always such that the counterweight of the front driver

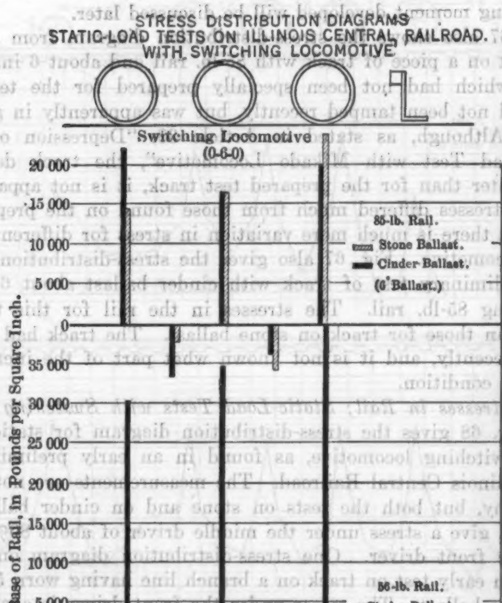

STRESS DISTRIBUTION DIAGRAMS
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD.
WITH SWITCHING LOCOMOTIVE
Switching Locomotive
(0-6-0)
Stress in Base of Rail, in Pounds per Square Inch.
20 000
15 000
10 000
5 000
0
35 000
30 000
25 000
20 000
15 000
10 000
5 000
0
−5 000
−10 000
−15 000
−20 000
−25 000
85-lb. Rail.
Stone Ballast.
Cinder Ballast.
(6" Ballast.)
56-lb. Rail.
Cinder Ballast.

FIG. 68.

was at its lowest point when this driver passed over the middle instrument of the three, the position of the counterweight of the second or other driver when this driver passed a given instrument was different from that of the first driver when its record was made. It is seen, then, that the tests were made in such a way as to give the same effect of counterweight in each run, but that the several drivers had different positions of counterweight when passing an instrument and making a record.

In all the moving load tests here reported, measurement of strain was made on the two sides of the rail by each instrument, and the average stress for the two sides of the rail is here reported. In some cases the record on one side or the other proved defective, and in these cases the good records were used. As there were four instruments, and generally four or five runs at each speed for each set-up, and three set-ups at adjacent locations, the value of the stress reported for a given driver at a given speed and for a given test section is generally the average of about a hundred records, and thus may be considered to be representative. It should be repeated that the stresses reported are the fiber stresses in the base of the rail at the mid-point of the gauge lines as reduced from the measured strains. As the instruments were placed between ties, the measurement of stress for positive moment occurred with the driver between ties. The measurement of maximum stress for a negative moment depended on the wheel spacing, occurring when the adjacent drivers of the Mikado locomotive were over ties and when the drivers of the Atlantic locomotive were between ties.

In Figs. 69 to 91 the data obtained in the tests on the Illinois Central Railroad are presented in graphical form for the three types of locomotive used. For each speed, the stress under each wheel has been plotted, and also the stress at the point of maximum negative moment between wheels. The scale of the stress is to be taken as beginning under the wheel for the positive moment stress and at the point between wheels indicated for the negative moment stress. Straight lines have been drawn to give the general trend of the stresses. The type of locomotive and descriptive items concerning the track and its condition are noted on the diagrams. The stresses indicated by the straight lines on these diagrams were considered to be representative of the average stresses for the given conditions. The averages of all the values given by the straight lines have been taken for two speeds, and have been plotted for both positive and negative moments on Figs. 92 to 97. As has already been noted, the type of instrument used in the 1915 tests is less accurate than the newer instrument, and gives smaller values than the true ones. For this reason, the 1916 results will be used for comparisons wherever possible.

46.—Stresses in Rail; Tests on D., L. & W. R. R.—Figs. 98 to 101 give speed-stress diagrams obtained from the data of the tests on the

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

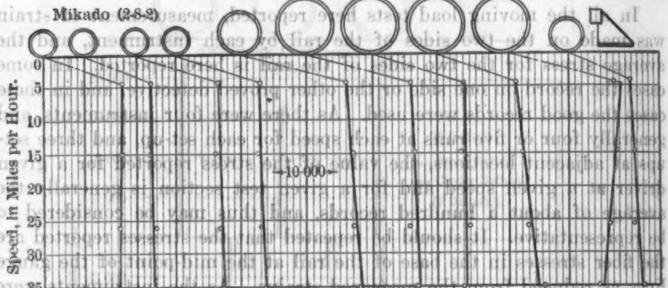


FIG. 69–a.

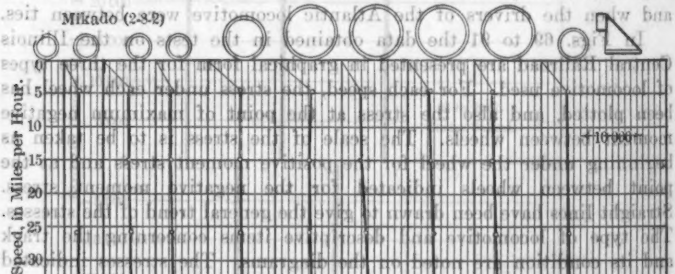


FIG. 69–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

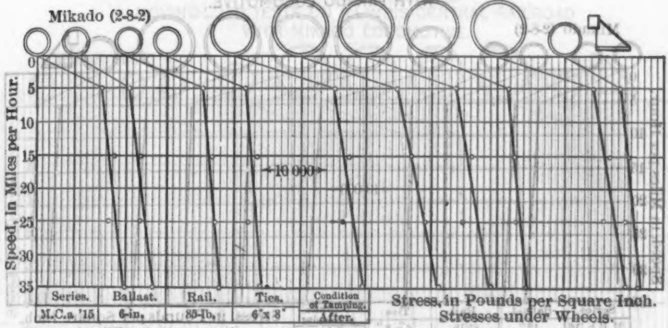


FIG. 70–a.

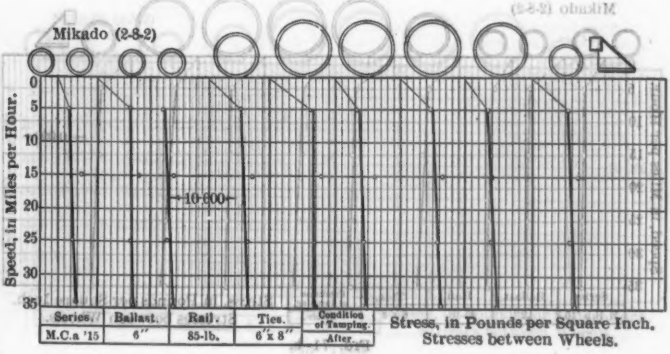
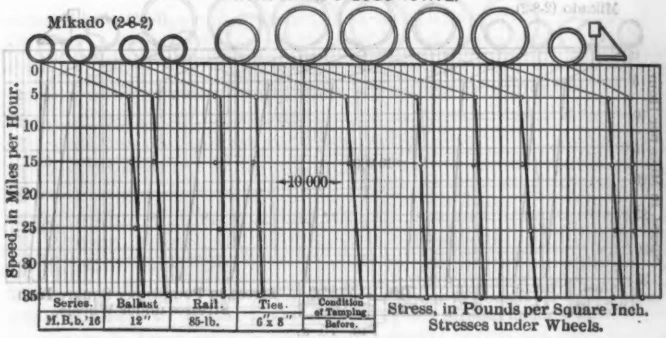


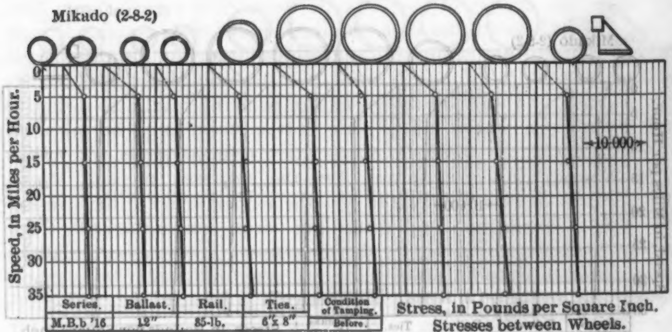
FIG. 70–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.



Series.	Ballast	Rail.	Ties.	Condition of Tamping.
M.B.b.'16	12"	85-lb.	6" x 8"	Before.

FIG. 71-a.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
M.B.b '16	12"	85-lb.	6" x 8"	Before.

FIG. 71-b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

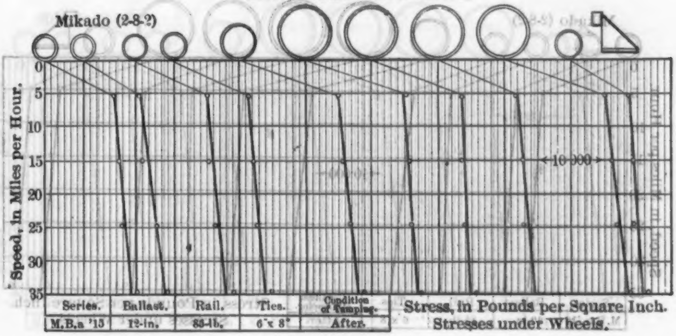


FIG. 72–a.

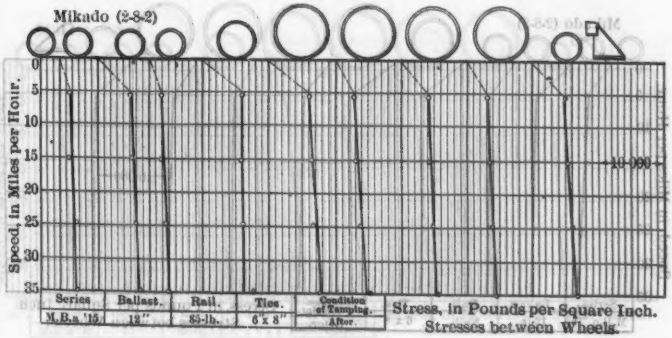


FIG. 72–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

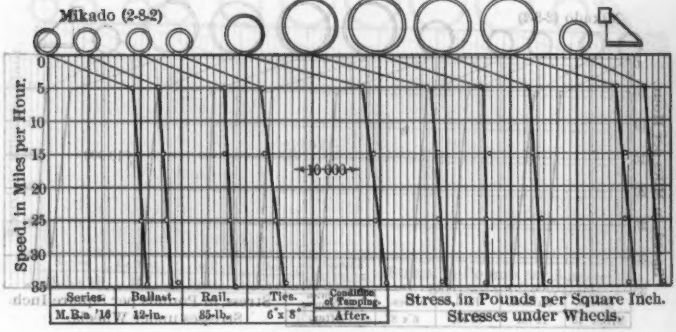


FIG. 73–a.

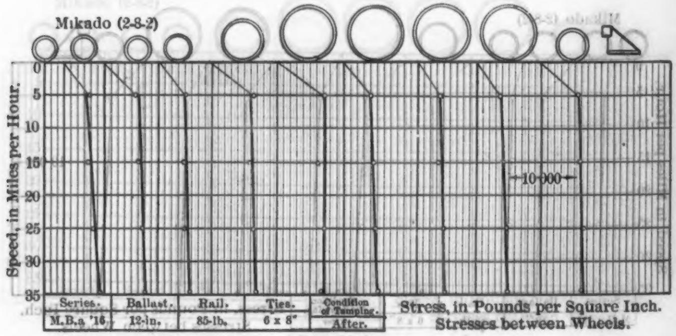


FIG. 73–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

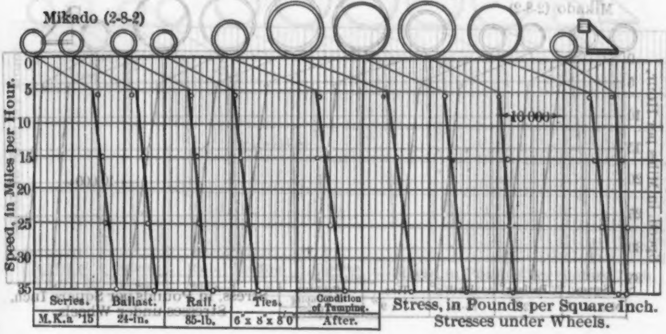


FIG. 74–a.

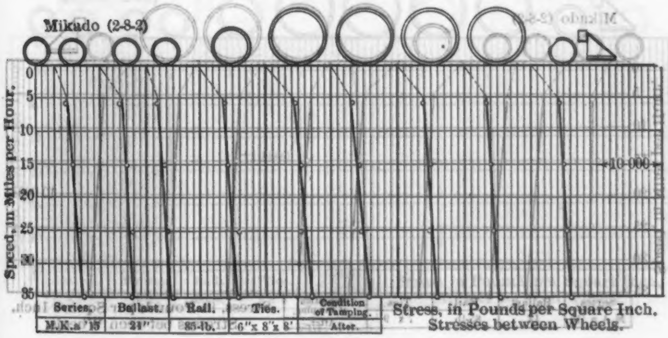


FIG. 74–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

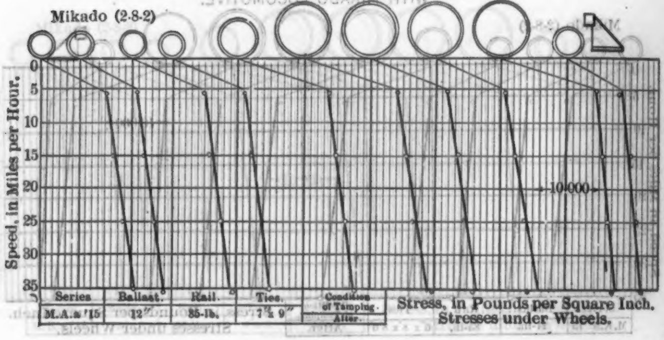


FIG. 75–a.

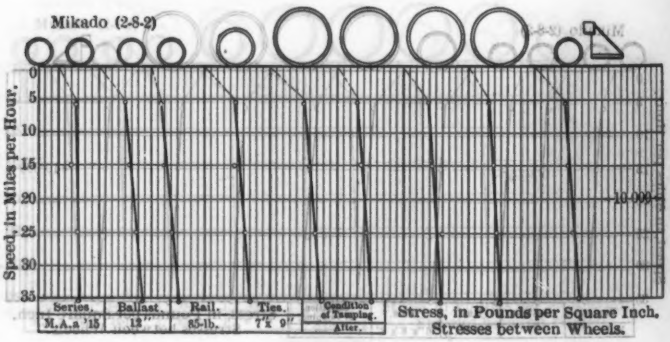


FIG. 75–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

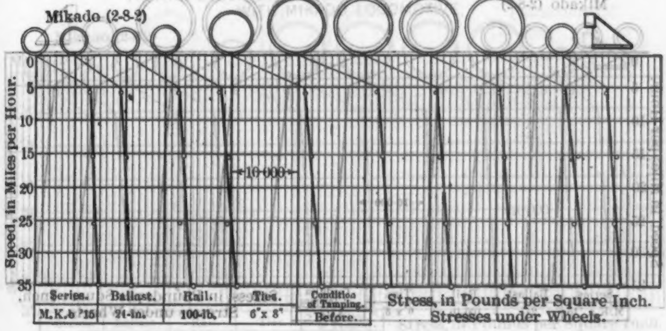


FIG. 76–a.

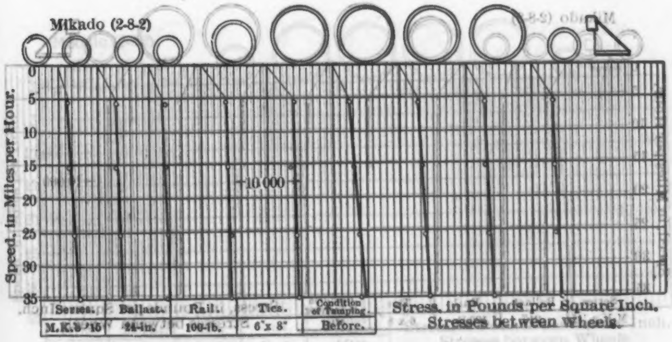


FIG. 76–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE

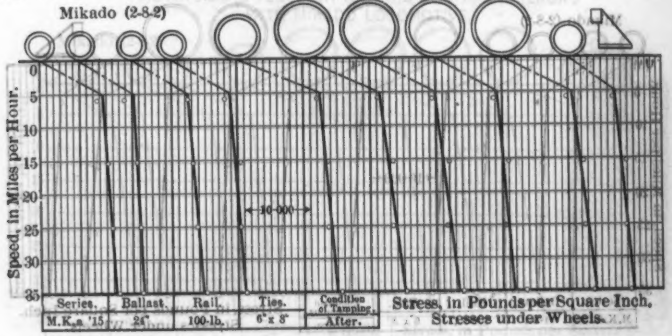


FIG. 77–a.

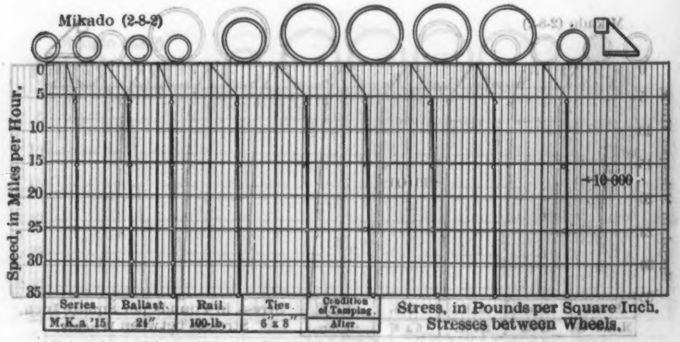


FIG. 77–b.

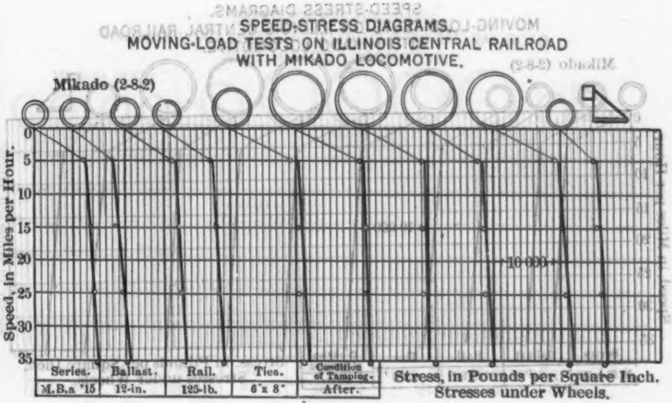


FIG. 78–a.

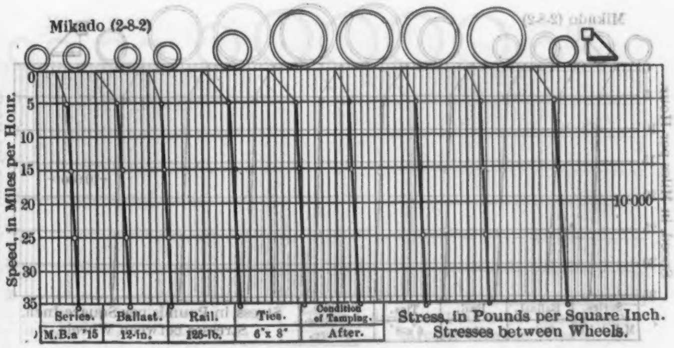


FIG. 78–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

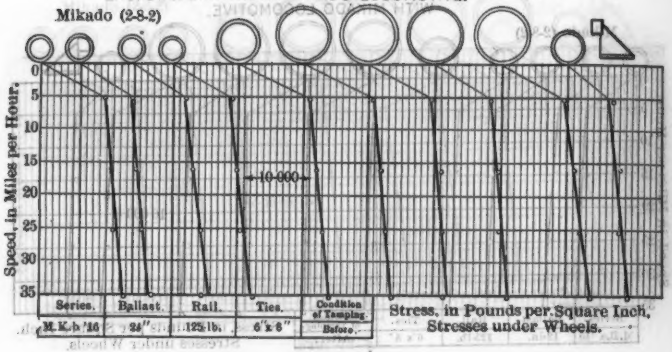


FIG. 79-a.

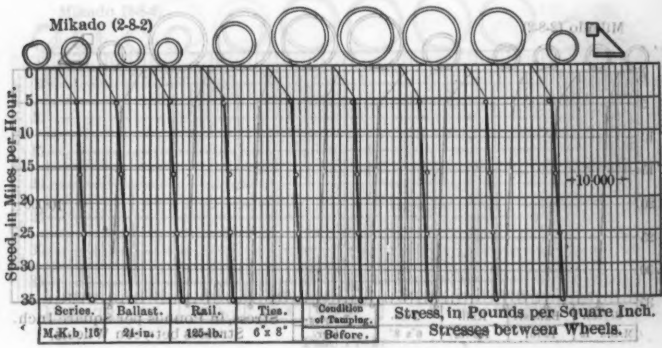


FIG. 79-b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

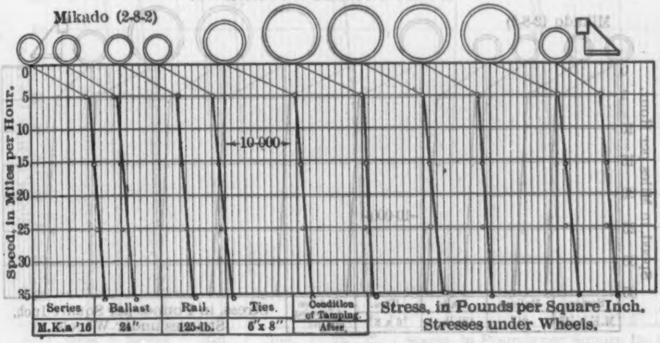


FIG. 80-a.

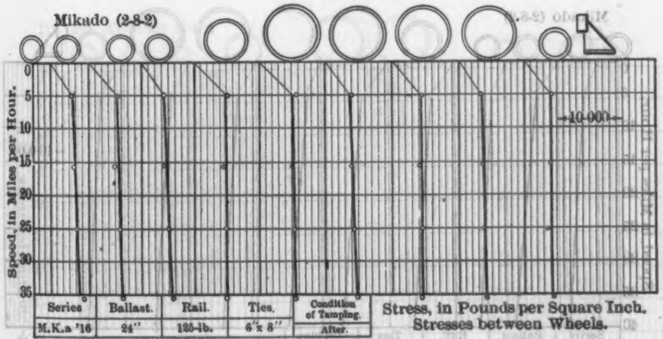


FIG. 80-b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

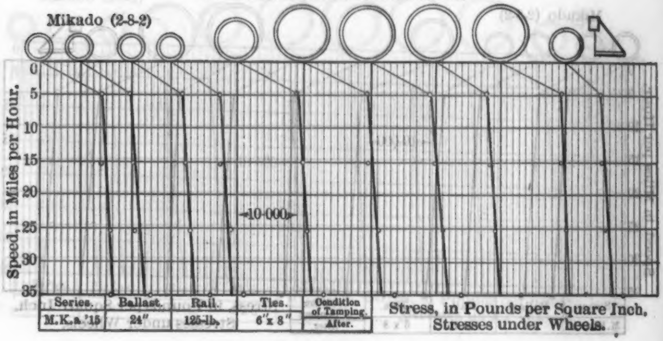


FIG. 81–a.

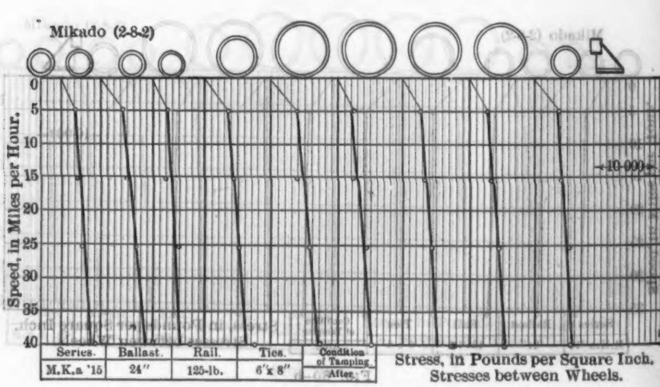
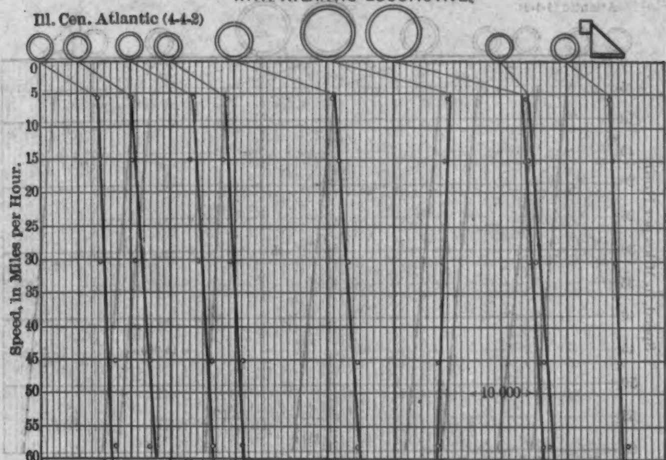


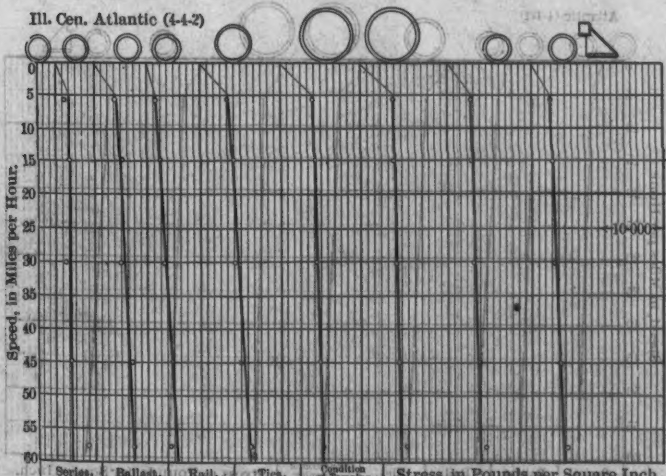
FIG. 81–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH ATLANTIC LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
A.C.a '15	6"	85-lb.	6" x 8"	After.

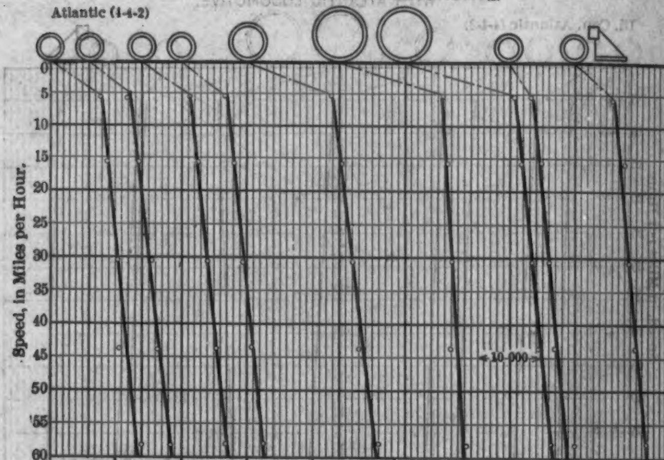
FIG. 82–a.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
A.C.a '15	6-in.	85-lb.	6' x 8'	After.

FIG. 82–b.

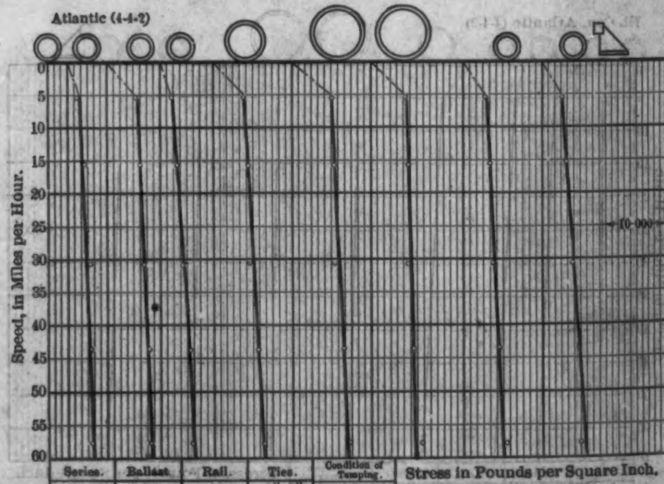
SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH ATLANTIC LOCOMOTIVE.



Series.	Ballast	Rail.	Ties.	Condition of Tamping.
A.B.a '16	12"	85-lb	6" x 8"	After.

Stress in Pounds per Square Inch.
Stresses under Wheels.

FIG. 83–a.



Series.	Ballast	Rail.	Ties.	Condition of Tamping.
A.B.a '16	12"	85-lb	6" x 8"	After.

Stress in Pounds per Square Inch.
Stresses between Wheels.

FIG. 83–b.

SPEED-STRESS DIAGRAMS
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH ATLANTIC LOCOMOTIVE.

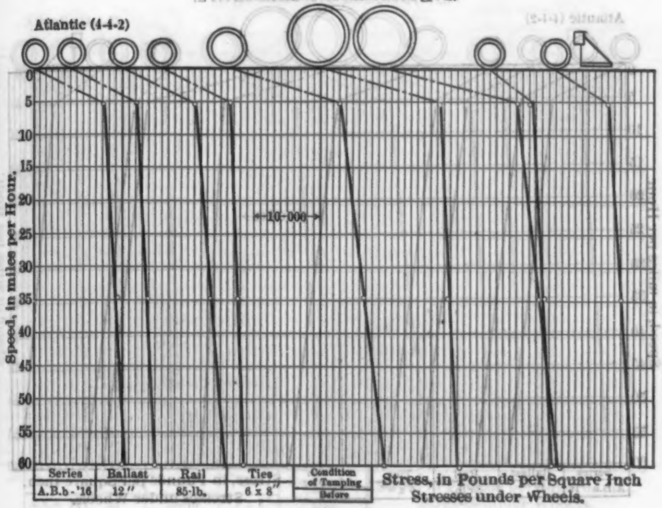


FIG. 84—a.

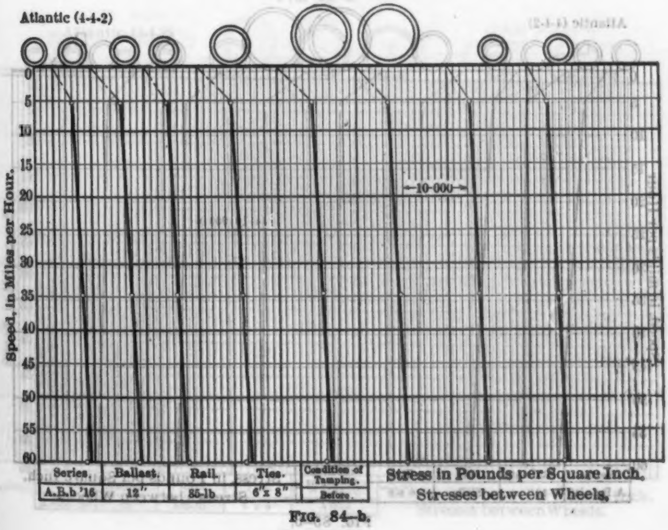


FIG. 84—b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH ATLANTIC LOCOMOTIVE.

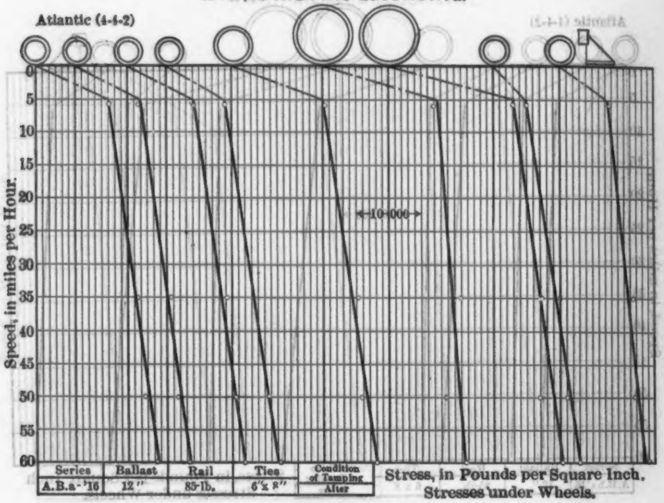


FIG. 85-a.

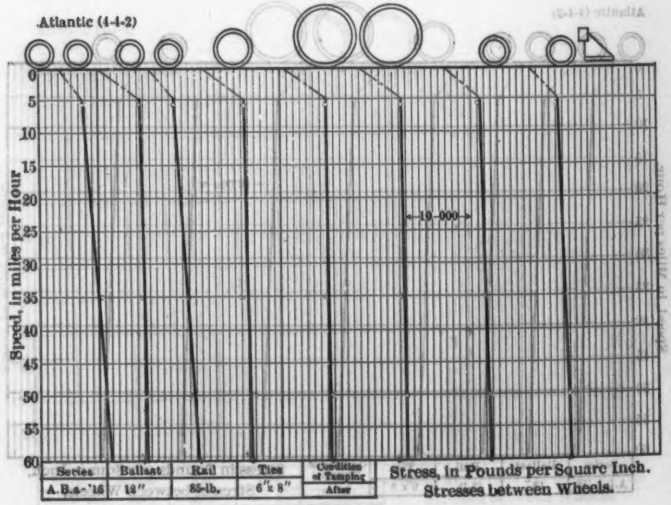
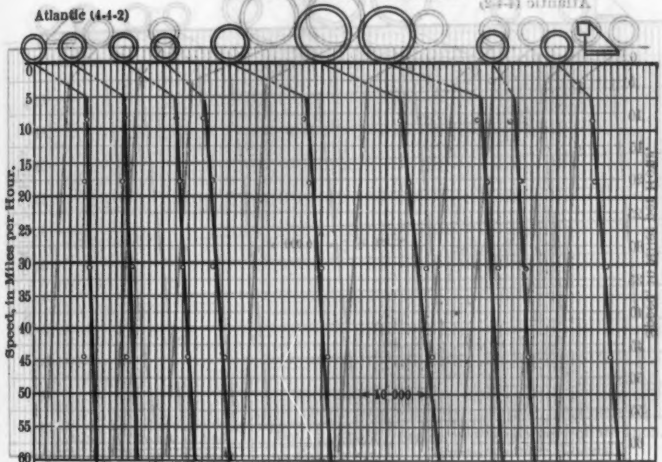


FIG. 85-b.

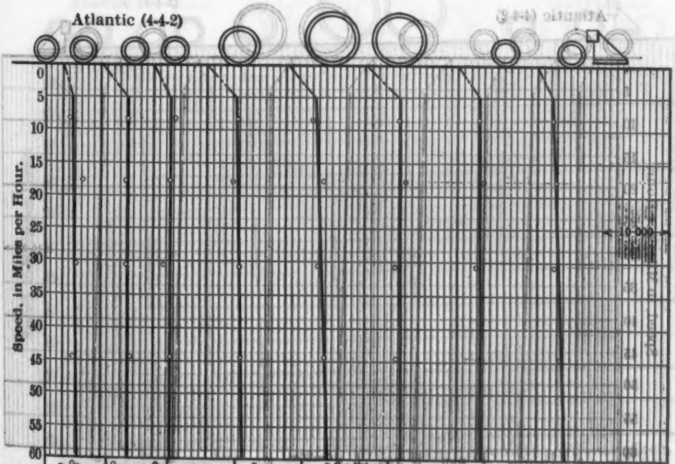
SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH ATLANTIC LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
A.K.a '15	24"	100-lb	6"x 8"	After.

Stress in Pounds per Square Inch.
Stresses under Wheels.

FIG. 86–a.



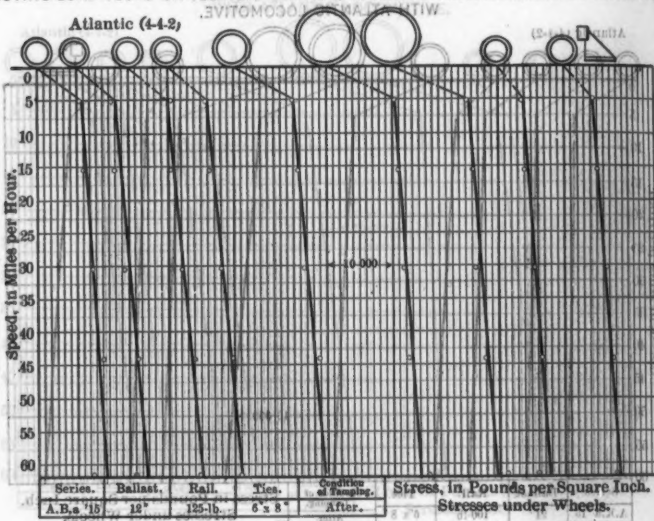
Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
A.K.a '15	24"	100-lb.	6"x 8"	After.

Stress in Pounds per Square Inch.
Stresses between Wheels.

FIG. 86–b.

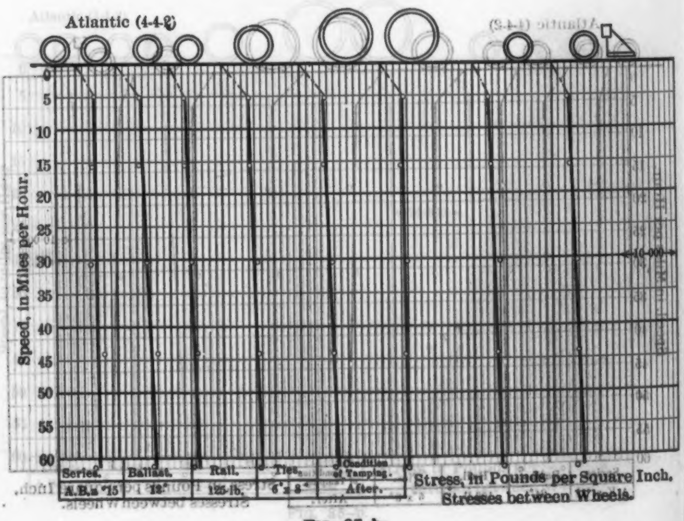
SPEED-STRESS DIAGRAMS.

MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH ATLANTIC LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
A.B,a '15	12"	125-lb.	6" x 8"	After.

FIG. 87–a.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
A.B,a '15	12"	125-lb.	6" x 8"	After.

FIG. 87–b.

SPEED-STRESS DIAGRAMS.

MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH ATLANTIC LOCOMOTIVE.

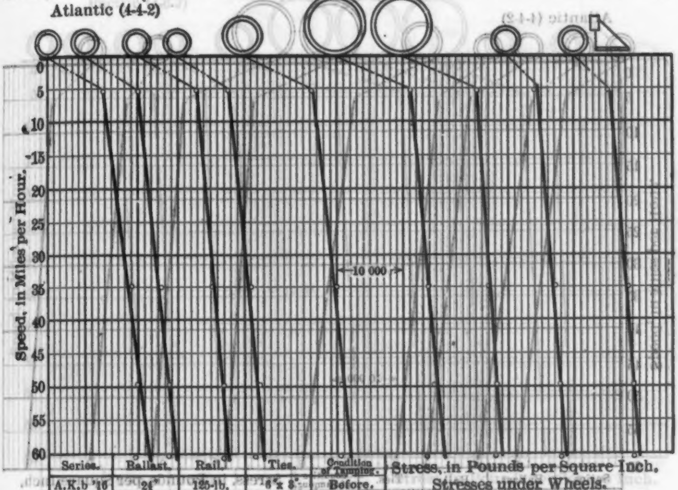


FIG. 88–a.

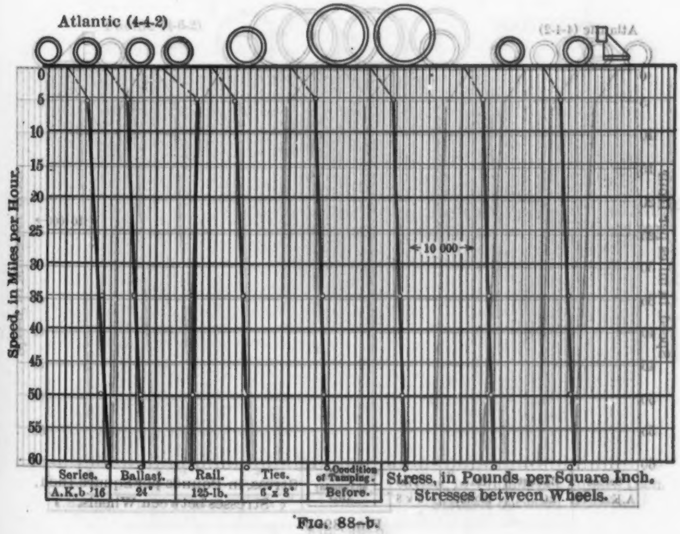
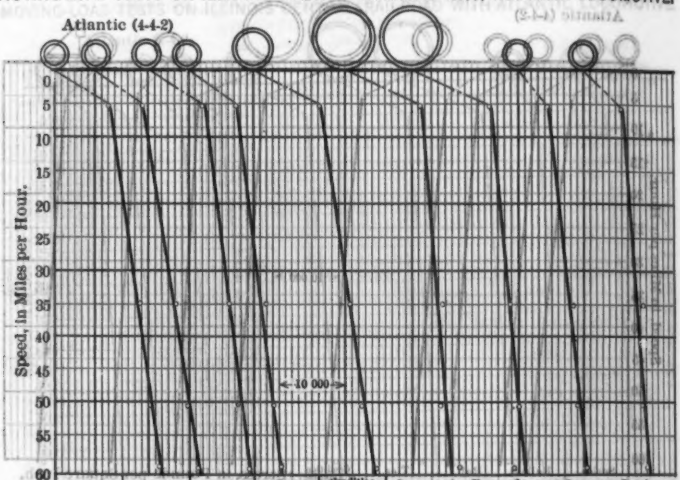


FIG. 88–b.

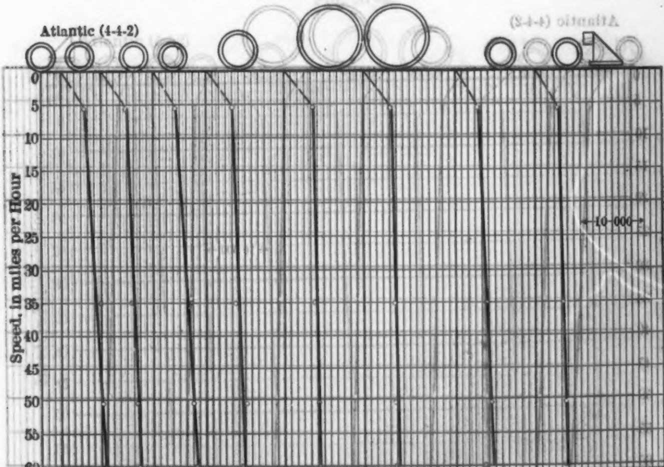
SPEED-STRESS DIAGRAMS.

MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH ATLANTIC LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
A.K.a '16	24"	125-lb.	6"x 8"	After.

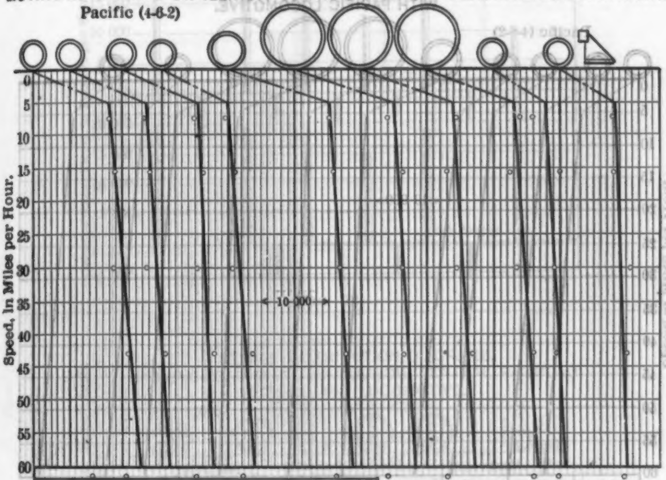
FIG. 89–a.



Series	Ballast	Rail	Ties	Condition of Tamping
A.K.a '16	24"	125-lb.	6"x 8"	After

FIG. 89–b.

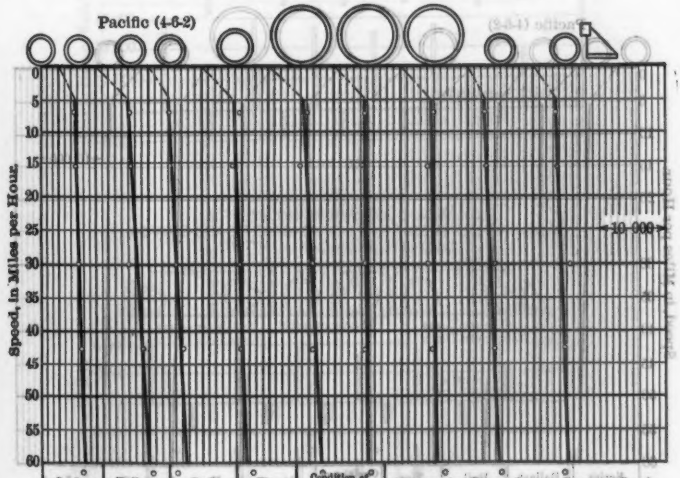
SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH PACIFIC LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
P.B.a '15	12"	85-lb.	6"x 8"	After.

Stress, in Pounds per Square Inch.
Stresses under Wheels.

Fig. 90–a.

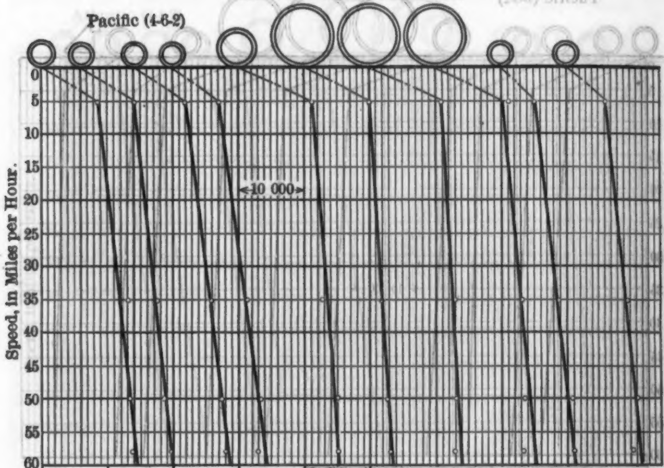


Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
P.B.a '15	12"	85-lb.	6"x 8"	After.

Stress, in Pounds per Square Inch.
Stresses between Wheels.

Fig. 90–b.

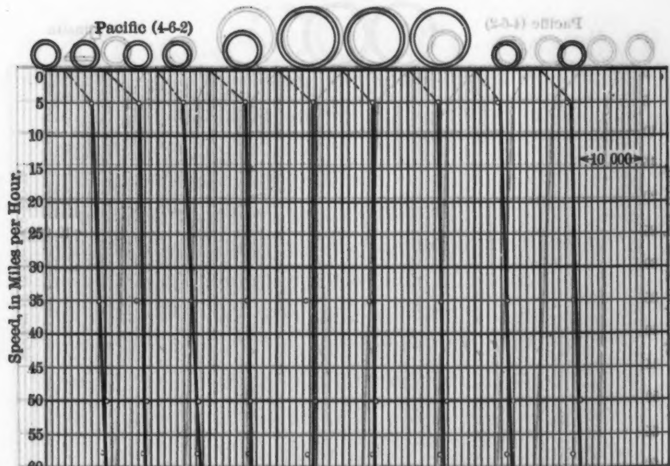
SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH PACIFIC LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
P.K.a '16	24"	125-lb.	6" x 8"	After.

Stress, in Pounds per Square Inch.
Stresses under Wheels.

FIG. 91–a.



Series.	Ballast.	Rail.	Ties.	Condition of Tamping.
P.K.a '16	24"	125-lb.	6" x 8"	After.

Stress, in Pounds per Square Inch.
Stresses between Wheels.

FIG. 91–b.

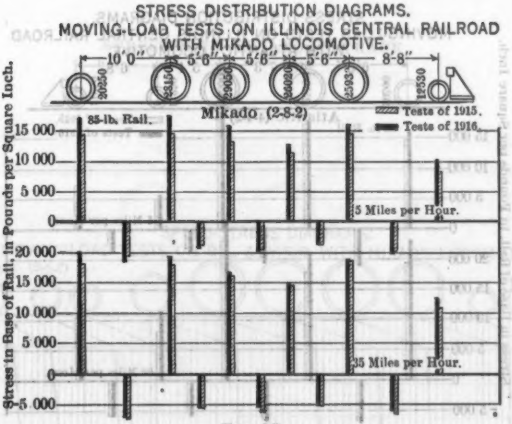


FIG. 92.

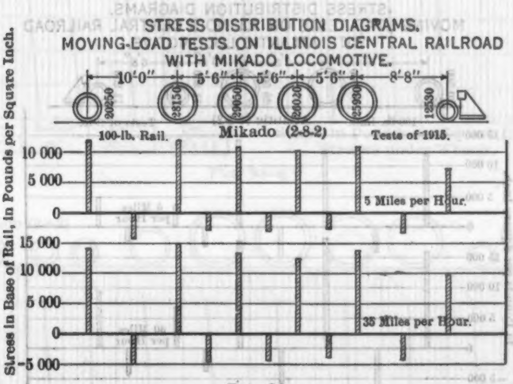


FIG. 93.

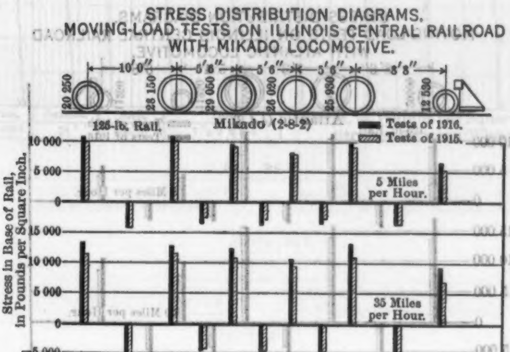


FIG. 94.

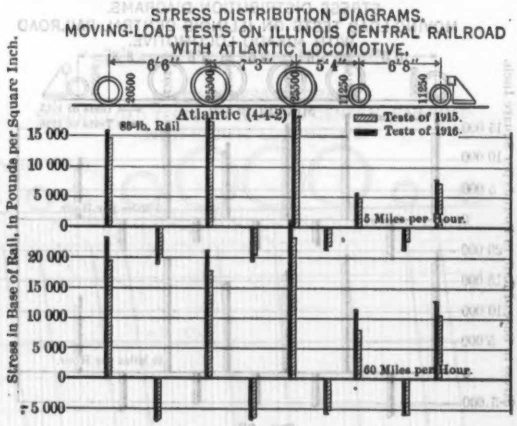


FIG. 95.

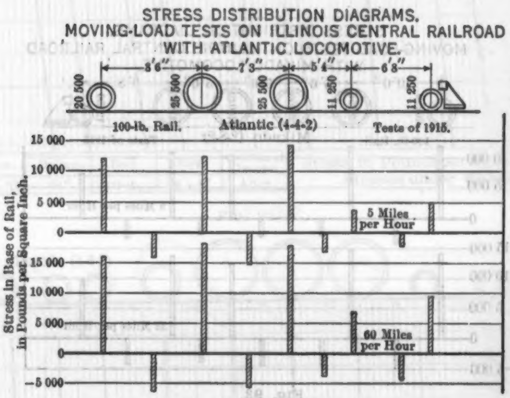


FIG. 96.

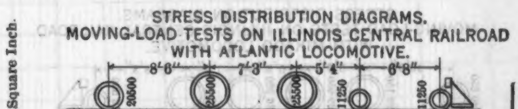
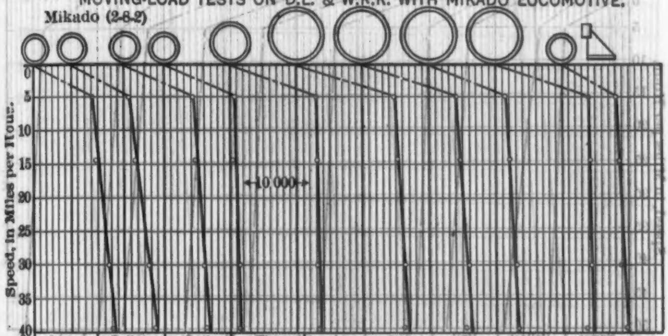


FIG. 97.

SPEED-STRESS DIAGRAMS.

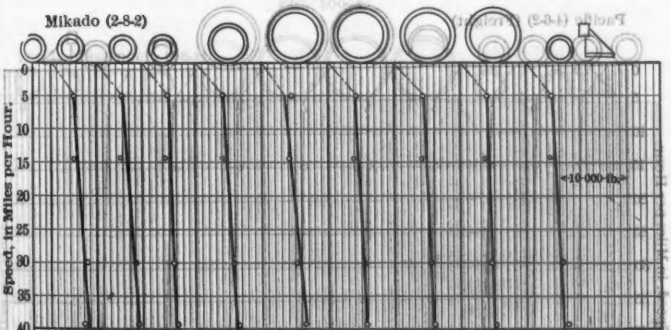
MOVING-LOAD TESTS ON D.L. & W.R.R. WITH MIKADO LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.
2061-2074	18"	105-lb.	7"x 9"x 8' 6"

Stress, in Pounds per Square Inch.
Stresses under Wheels.

FIG. 98–a.

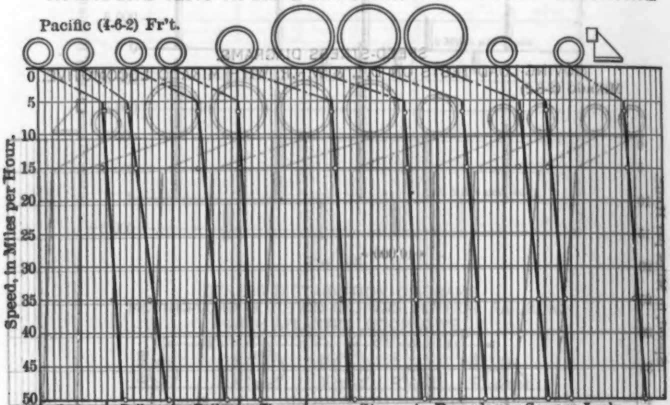


Series.	Ballast.	Rail.	Ties.
2061-2074	18"	105-lb.	7"x 9"x 8' 6"

Stress, in Pounds per Square Inch.
Stresses between Wheels.

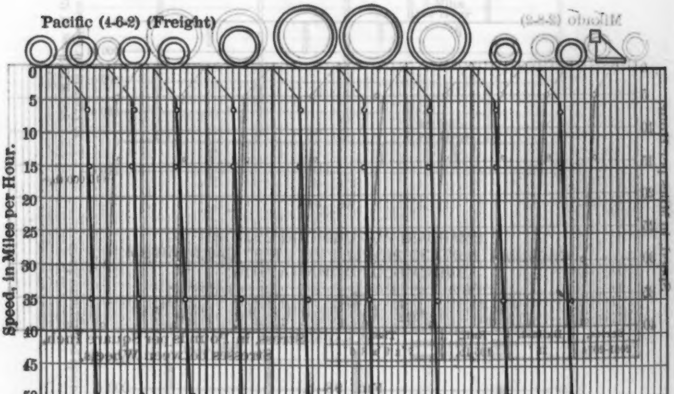
FIG. 98–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON D.L. & W.R.R. WITH PACIFIC FREIGHT LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.
2032-2045	18"	105-lb.	7" x 9" x 8' 6"

FIG. 99–a.



Series.	Ballast.	Rail.	Ties.
2032-2045	18"	105-lb.	7" x 9" x 8' 6"

FIG. 99–b.

SPEED-STRESS DIAGRAMS.
MOVING-LOAD TESTS ON D.L. & W.R.R. WITH PACIFIC PASSENGER LOCOMOTIVE.

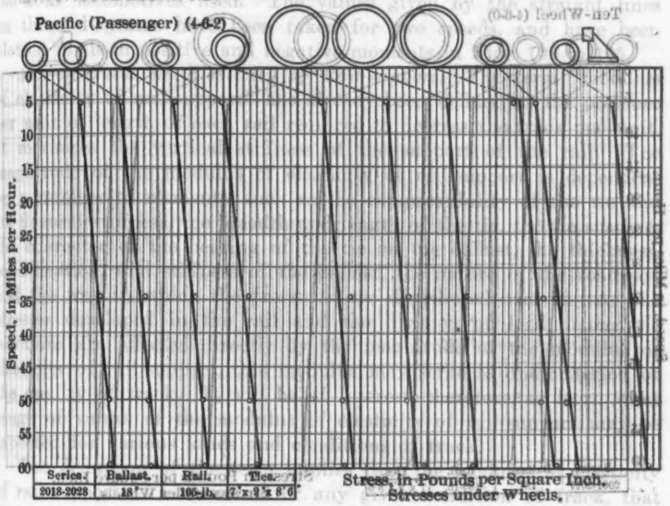


FIG. 100-a.

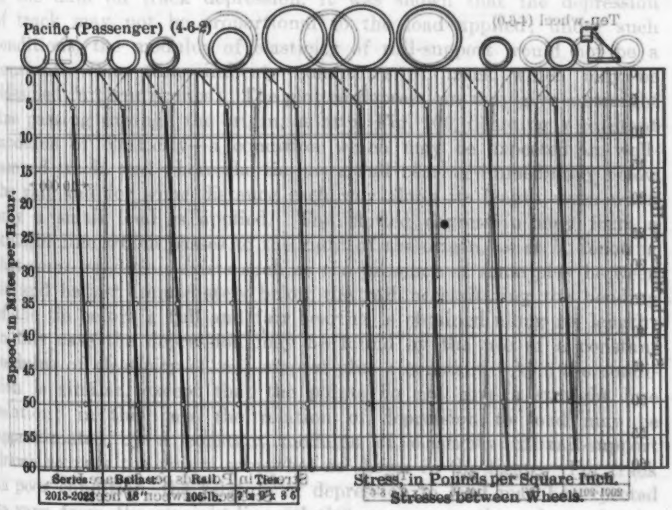


FIG. 100-b.

SPEED-STRESS DIAGRAMS.

MOVING-LOAD TESTS ON D.L. & W.R.R. WITH TEN-WHEEL LOCOMOTIVE.

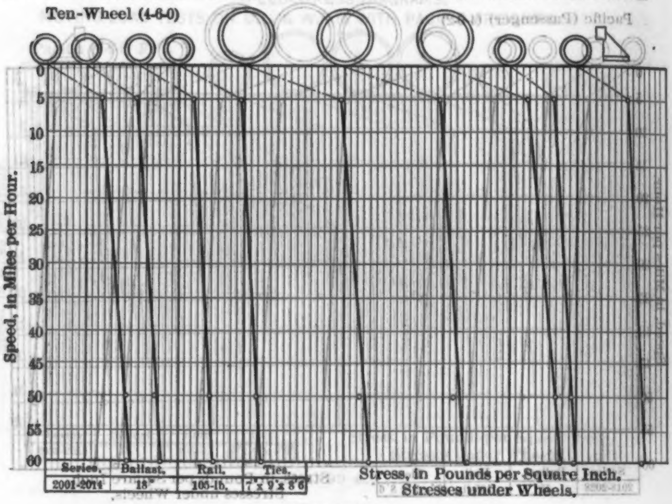


FIG. 101-a.

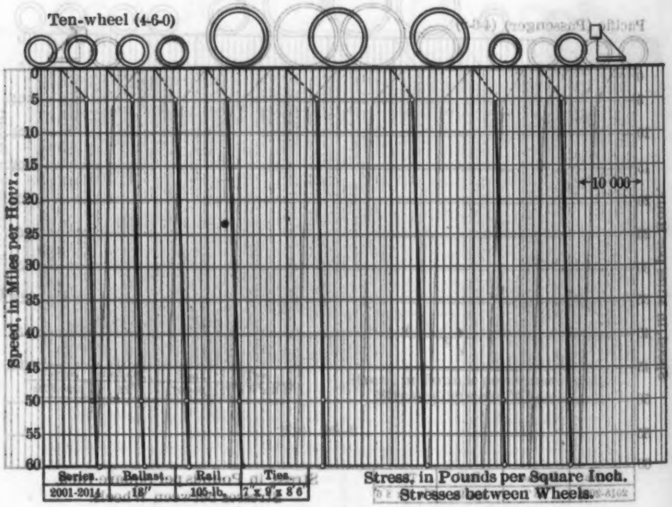


FIG. 101-b.

Delaware, Lackawanna and Western Railroad near Dover, N. J., for the four locomotives used. The values given by the straight lines on these diagrams have been taken for two speeds, and have been plotted for both positive and negative moments in Figs. 102 to 105.

47.—*Modulus of Elasticity of Rail-support.*—The term "modulus of elasticity of rail-support" has already been defined as the pressure per unit of length of each rail required to depress the track one unit. It measures the vertical stiffness of the support of the rail. The magnitude of the modulus of elasticity of rail-support is dependent on a number of elements, such as the compressibility of the tie and its flexural stiffness, the breadth and length of the tie, the tie spacing, the character of the bearing of the tie on the ballast, the thickness, solidification, and stiffness of the ballast, the nature of the roadway, and the way in which the pressures are distributed over it. As the stresses developed in the rail and the division of load among tie reactions are affected directly by the magnitude of the modulus of elasticity of rail-support, this modulus may serve as one criterion of the quality of track. It will be of interest, therefore, to learn what range of values of the modulus of elasticity of rail-support may be expected for various kinds and conditions of track.

The definition of the term implies that the modulus of elasticity of rail-support is a constant for any given condition of track, that the track has the usual property of an elastic body. In the discussion of the data on track depression, it was shown that the depression of track may not be proportional to the load applied; under such conditions, the modulus of elasticity of rail-support would not be a constant. The tests indicate conditions of track which may be classified in three groups: The plotted depressions may give a straight line passing through the origin, as at *a*, Fig. 106, denoting a constant modulus of elasticity—a condition which may be expected in well-tamped track, and where the tie has a full bearing immediately below the rail and for some distance each way along its length, even when only a small load is applied. The plotted depression may form a straight line which passes to the right of the origin, as at *b*, denoting that some condition exists such as the presence of some play between tie and ballast immediately below the rail necessitating the bending of the tie before a full and fair bearing is obtained along its length; in this case the depression may be found by the use of a constant modulus of elasticity of rail-support plus a constant. At *c* the condition is similar, except that the points do not give a straight line relation; in this case the relation of depression to load may be approximated by a constant modulus of elasticity of rail-support drawn to pass the points somewhat as shown in the figure. For track in poor condition, the relation of depression to load may be expected to vary from the straight-line relation even more than shown in the

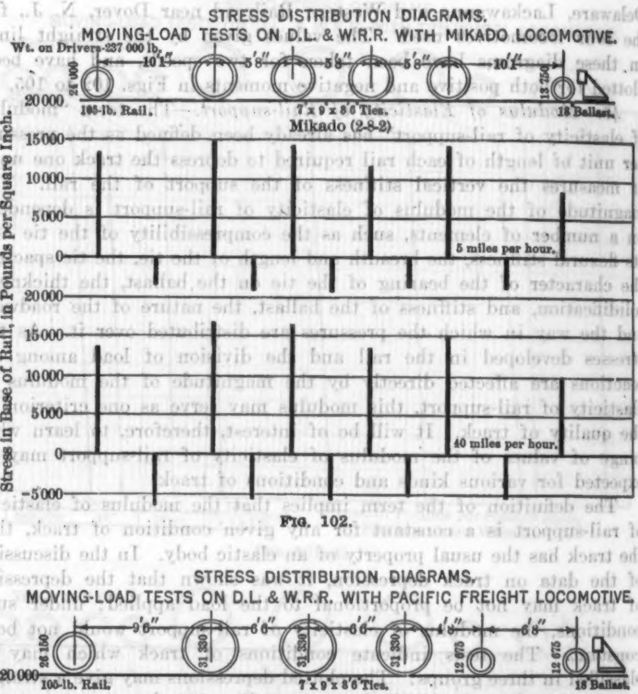


FIG. 102.

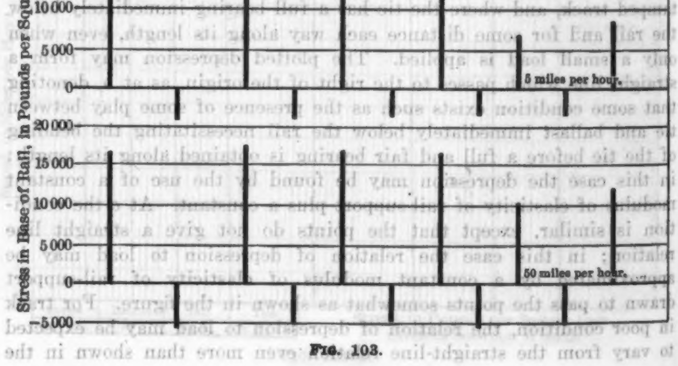


FIG. 103.

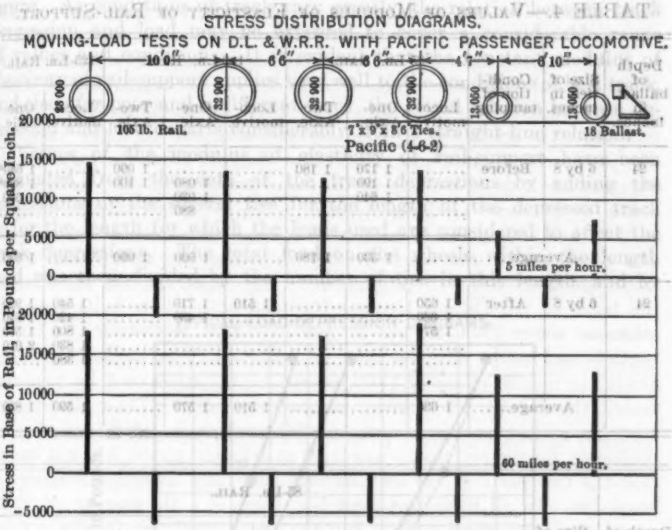


FIG. 104.

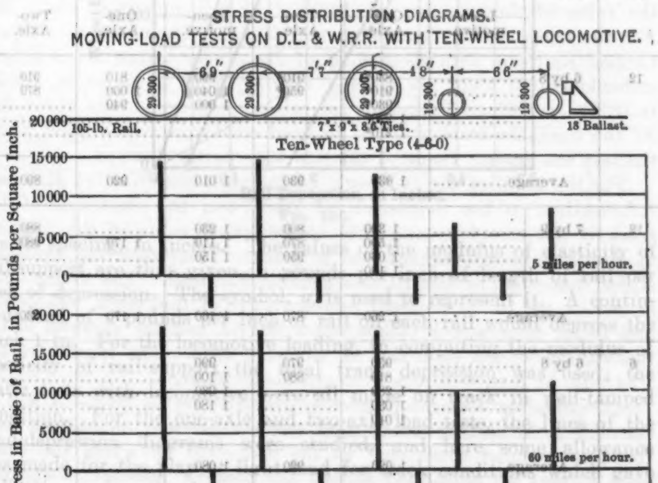


FIG. 105.

TABLE 4.—Values of Modulus of Elasticity of Rail-Support.

Depth of ballast, in inches.	Size of ties, in inches.	Condition of tamping.	85-Lb. Rail. Locomotive.	85-Lb. Rail. One-Axle.	85-Lb. Rail. Two-Axle.	100-Lb. Rail. Locomotive.	100-Lb. Rail. One-Axle.	100-Lb. Rail. Two-Axle.	125-Lb. Rail. Locomotive.	125-Lb. Rail. One-Axle.
24	6 by 8	Before	....	1 170	1 180	....	....	1 090	....	1 640*
			....	1 190	....	....	1 080	1 100	....	1 820*
			....	1 640	....	....	1 030	....	....	1 600
			....	....	....	....	880	....	....	....
		Average......	....	1 330	1 180	....	1 000	1 090	....	1 690
24	6 by 8	After	1 650	....	....	1 510	1 710	....	1 540	1 920
			1 660	....	....	....	1 430	....	1 420	1 840
			1 570	....	....	....	....	....	1 800	1 560
			....	....	....	....	....	....	1 830	2 010
			....	....	....	....	....	....	1 380	....
		Average......	1 630	....	....	1 510	1 570	....	1 590	1 830

Depth of ballast, in inches.	Size of ties, in inches.	85-Lb. Rail. Before Tamping. Locomotive.	85-Lb. Rail. Before Tamping. One-Axle.	85-Lb. Rail. Before Tamping. Two-Axle.	85-Lb. Rail. After Tamping. Locomotive.	85-Lb. Rail. After Tamping. One-Axle.	85-Lb. Rail. After Tamping. Two-Axle.
12	6 by 8	....	830*	910*	990	810	910
		....	910*	950*	1 040	1 000	870
		....	930*	....	1 000	940	....
		....	1 270*	....	....	....	....
		....	1 210*	....	....	....	....
	Average............		1 030	930	1 010	920	890
12	7 by 9	....	1 300	800	1 230	....	880
		....	1 500	870	1 110	1 170	980
		....	1 060	950	1 150	....	....
		....	1 190	....	....	....	....
	Average............		1 260	870	1 160	1 170	930
6	6 by 8	....	950	970	990	....	....
		....	810	880	1 100	....	....
		....	1 280	....	1 030	....	....
		....	1 030	....	1 180	....	....
		....	1 010	....	....	....	....
	Average............		1 020	920	1 060	....	....

* This track, marked as "before tamping," seemed to be in as good condition as "after tamping."

figure. As conditions of track vary greatly, the relation between track depression and load may be expected to cover a considerable range of values and conditions. It is evident that the constant modulus of elasticity of rail-support applies very well to the conditions of best track, and that, for poor and indifferent track, the actual relation between depression and load departs considerably from a straight-line relation.

Values of the modulus of elasticity of rail-support have been calculated from the data of the track depressions by adding the depressions at the several ties for the length of the depressed track or for the length for which the loads used are considered to affect the track depressions. The total load on the wheels within the length used was then divided by the number of ties in this length, and by

TYPICAL LOAD-DEPRESSION DIAGRAMS.

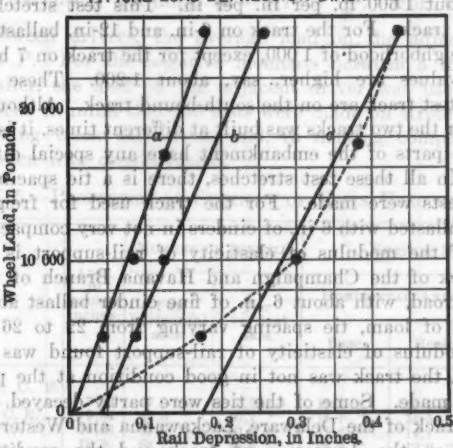


Fig. 106.

the tie spacing, in inches. The values of the modulus of elasticity of rail-support are thus given in pounds per inch of length of rail per inch of depression. The symbol, u, is used to represent it. A continuous load of u pounds per inch of rail on each rail would depress the track 1 in. For the locomotive loading, in computing the modulus of elasticity of rail-support the total track depression was used; the static tests with locomotive were all made on track in well-tamped condition. For the one-axle and two-axle load tests, the lines of the load-depression diagrams were studied, and here some allowance was made for the play at light load for track conditions which gave indications of such play, net track depressions being used for the calculations. In some cases a general compromise straight line was

used. The values reported are given tentatively, and may need empirical modifications later in order to fit into other experimental data.

Table 4 gives the values of the modulus of elasticity of rail-support as calculated from the track depressions. The conditions of track are not stated very definitely, but, even in track marked "before tamping", the track was in good surface, and only in a few instances was it in need of tamping. The values derived from the tests for the different methods of loading on the same track agree very well. There seems to be some tendency toward a higher value of the modulus in the track having the heaviest rail. It is apparent that the character and condition of the track greatly influence the magnitude of the modulus of elasticity of rail-support. The value for the modulus on the track of the Illinois Central Railroad with 24-in. ballast may be taken as about 1 600 lb. per in. per in. This test stretch is on the north-bound track. For the track on 6-in. and 12-in. ballast, the values are in the neighborhood of 1 000, except for the track on 7 by 9-in. ties, where the values are higher, say, about 1 200. These last-named stretches of test track are on the south-bound track. Although the embankment for the two tracks was built at different times, it is not known that the two parts of the embankment have any special differences in condition. In all these test stretches, there is a tie spacing of 22 in. where the tests were made. For the track used for freight service, which was ballasted with 6 in. of cinders in not very compact condition, the value of the modulus of elasticity of rail-support is about 750. For the track of the Champaign and Havana Branch of the Illinois Central Railroad, with about 6 in. of fine cinder ballast above a light embankment of loam, tie spacing varying from 22 to 26 in. (56-lb. rail), the modulus of elasticity of rail-support found was about 530. At the time, the track was not in good condition at the point where the test was made. Some of the ties were partly decayed.

For the track of the Delaware, Lackawanna and Western Railroad, information on the depression of track and the condition of the track is not complete, and only an estimated value of the modulus of elasticity of rail-support can be given. This track was evidently stiffer than that of the Illinois Central Railroad. The value, 2 200 lb. per in. per in., is probably representative of this track. The track had 18 in. of trap rock ballast below the tie, and the material of the roadway below was such that it was very solid. The spacing of the 7 by 9-in. by 8 ft. 6-in. ties averaged about 22 in.

48.—Bending Moment Coefficient and Stresses in Rail.—It will be convenient, in making calculations and comparisons, to use the term "bending moment coefficient" as the coefficient by which the wheel load, *P*, may be multiplied to find the bending moment in the rail at a given point. Generally, the term will refer to the bending moment directly under a wheel load. In the case of a combination of wheels,

the moment under a wheel will be expressed in terms of the wheel load at the given point. The bending moment coefficient will also be used in connection with the maximum negative bending moment. In this case, the wheel load used in connection with the coefficient should be specified. The symbol, K, will represent the bending moment coefficient. The sections of the rails used in the tests are shown in Fig. 107. The properties of the sections are given in Table 5. In columns marked S, in Table 6, are recorded fiber stresses in base of rail derived from the tests on the Illinois Central Railroad, the values being averages of those plotted in Figs. 69 to 91. The stresses are given in thousands of pounds per square inch. Corresponding values of the bending moment coefficients are given in the columns to the right of the stresses (marked K). These were calculated from the stresses by the usual formula, $M = KP = \frac{SI}{c}$. For the negative moments between drivers, the average load on the adjacent drivers was used as P. For the negative moments between a driver and a truck or trailer, the load on the adjacent driver was used as P. It should be borne in mind that the tests were made on stretches of track having differing track conditions, and, in making comparisons, this should be taken into consideration.

TABLE 5.—PROPERTIES OF SECTIONS OF RAILS USED IN TESTS.

Rail section.	Area, in square inches.	Moment of Inertia.		Section Modulus.		
		For horizontal axis.	For vertical axis.	Horizontal Axis.		Vertical Axis.
				Base.	Head.	Base.
85-lb. Am. Soc. C. E. (worn)	7.85	27.0-in.⁴	7.3-in.⁴	11.1-in.³	10.1-in.³	2.79-in.³
100-lb. Am. Soc. C. E.	9.84	44.0 "	10.7 "	16.1 "	14.5 "	3.72 "
105-lb. D. L. & W.	10.23	47.6 "	10.0 "	17.2 "	14.7 "	3.72 "
125-lb. P. R. R.	12.30	68.7 "	13.4 "	22.9 "	19.6 "	4.87 "

Table 7 gives stresses and bending moment coefficients obtained in a similar manner from the tests on the track of the Delaware, Lackawanna and Western Railroad.

The bending moment coefficients found in the tests with the several types of locomotive, given in the foregoing tables, are shown diagramatically in Figs. 108 to 117.

In Fig. 118 the stresses in the rail at speeds of 5 miles per hour and 35 or 60 miles per hour, taken from Table 6, are plotted for the three sections of rail.

Discussion of the results given in these tables and diagrams will be found in the following pages.

TABLE 6.—(*Continued.*)

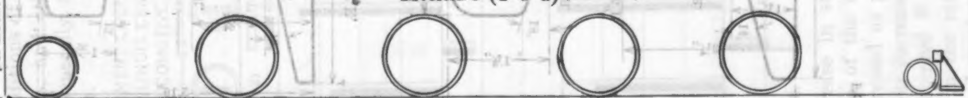
ATLANTIC (4–4–2)

Weight of rail, in pounds per yard.	Year.	Speed, in miles per hour.	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K
85	1915	5	14.6	7.9	−5.0	−2.2	17.4	7.5	−4.7	−2.1	18.4	7.9	−3.0	−1.3	4.8	4.7	−2.4	−2.4	7.1	7.0
		60	19.4	10.6	−7.4	−3.2	17.7	7.7	−6.5	−2.8	23.2	10.1	−5.2	−2.3	8.0	7.9	−5.8	−5.7	10.4	10.3
85	1916	5	15.9	8.6	−6.1	−2.7	17.9	7.8	−5.5	−2.4	19.6	8.5	−3.8	−1.7	5.7	5.6	−3.6	−3.6	7.9	7.8
		60	23.3	12.6	−7.4	−3.3	21.2	9.3	−7.0	−3.2	26.1	11.4	−5.7	−2.5	11.5	11.2	−5.8	−5.8	12.8	12.9
100	1915	5	11.8	9.3	−4.3	−2.8	12.4	7.8	−5.2	−3.4	14.4	9.2	−3.2	−2.1	3.6	5.1	−2.2	−3.3	5.0	7.2
		60	15.7	12.8	−6.5	−4.1	18.2	11.5	−5.5	−3.5	17.7	11.2	−3.7	−2.3	6.6	9.5	−4.4	−6.3	9.5	13.7
125	1915	5	9.1	10.1	−3.7	−3.4	10.4	9.4	−3.3	−3.2	11.6	10.4	−2.5	−2.3	4.0	8.0	−2.5	−5.3	4.7	9.5
		60	13.6	14.7	−5.4	−4.8	13.9	12.5	−4.0	−3.6	15.6	14.0	−4.2	−3.8	7.6	15.5	−4.6	−9.4	8.5	17.3
125	1916	5	10.3	11.5	−4.1	−3.7	11.3	10.1	−3.9	−3.5	11.8	10.6	−3.2	−3.0	4.5	9.2	−3.4	−7.0	6.0	12.2
		60	17.2	19.2	−5.6	−5.1	15.8	14.2	−5.4	−4.9	15.9	14.2	−5.2	−4.7	9.8	19.7	−5.0	−10.3	10.4	21.2

PACIFIC (4–6–2)

Weight of rail, in pounds per yard.	Year.	Speed, in miles per hour.	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K
85	1915	5	15.3	7.7	−5.3	−2.4	15.0	6.7	−4.5	−1.9	13.8	5.7	−4.5	−1.9	13.1	5.8	−2.5	−1.1	7.8	7.0	−3.5	−3.1	8.5	7.7
		60	18.9	9.5	−7.9	−3.5	18.2	8.1	−4.8	−2.1	17.2	7.1	−5.2	−2.2	16.7	7.4	−4.5	−2.0	10.7	9.7	−5.0	−4.5	10.1	9.1
125	1916	5	11.3	11.7	−5.6	−5.1	10.1	9.3	−4.7	−4.1	10.9	9.2	−4.7	−4.1	10.2	9.3	−3.8	−3.5	5.2	9.6	−5.0	−9.3	6.1	11.8
		60	15.3	15.9	−5.5	−5.0	13.5	12.3	−5.0	−4.4	14.0	11.8	−5.3	−4.7	14.9	13.6	−5.7	−5.2	11.7	21.6	−6.3	−11.7	11.6	21.5

TABLE 6.—STRESSES IN RAILS, *S*, AND BENDING MOMENT COEFFICIENTS, *K*, FROM MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD.



Weight of rail, in pounds per yard.	Year.	Speed, in miles per hour.	Mikado (2-8-2)																					
			S	*K*	*S*	*K*	*S*	*K*	*S*	*K*	*S*	*K*	*S*	*K*	*S*	*K*	*S*	*K*	*S*	*K*	*S*	*K*	*S*	*K*
85	1915	5	14.8	7.9	−5.6	−2.3	14.7	5.8	−3.5	−1.4	18.3	5.0	−4.4	−1.8	11.4	4.9	−3.5	−1.5	14.7	6.4	−4.7	−2.0	8.4	7.7
		35	18.1	9.9	−7.4	−2.8	17.9	7.1	−5.5	−2.1	16.1	6.1	−6.1	−2.3	14.7	6.4	−5.1	−2.1	18.6	7.8	−6.6	−2.6	10.9	9.7
85	1916	5	17.1	9.4	−6.6	−2.6	17.7	7.0	−4.0	−1.6	16.0	6.1	−4.7	−1.9	12.9	5.5	−3.5	−1.5	16.1	6.9	−5.1	−2.2	10.3	9.1
		35	20.1	11.0	−7.0	−2.7	19.4	7.6	−5.3	−2.1	16.8	6.4	−5.3	−2.1	15.0	6.4	−5.0	−2.1	18.9	8.1	−6.1	−2.6	12.4	11.0
100	1915	5	12.0	9.6	−4.2	−2.5	12.0	6.9	−2.8	−1.7	10.8	5.9	−3.0	−1.9	10.1	6.2	−2.6	−1.7	10.8	6.7	−3.8	−2.2	7.2	9.2
		35	14.2	11.3	−4.8	−2.7	14.8	8.5	−4.6	−2.6	13.4	7.4	−4.4	−2.6	12.4	7.7	−3.7	−2.3	13.9	8.6	−4.4	−2.7	9.7	12.5
125	1915	5	9.8	11.0	−4.2	−3.5	10.1	8.1	−2.3	−2.0	9.4	7.3	−2.7	−2.4	8.0	7.0	−2.5	−2.3	9.4	8.4	−3.5	−3.3	5.5	10.0
		35	11.5	13.0	−6.1	−5.0	11.6	9.4	−4.3	−3.4	11.0	8.7	−4.4	−3.6	9.8	8.6	−4.4	−3.9	11.1	9.8	−5.8	−4.7	7.0	12.8
125	1916	5	10.9	12.3	−4.2	−3.4	10.8	8.8	−3.4	−2.7	9.7	7.6	−3.6	−3.0	8.3	7.3	−3.2	−2.8	10.1	8.9	−3.5	−3.1	6.6	12.0
		35	13.4	15.1	−5.1	−4.1	12.9	10.5	−4.3	−3.4	12.4	9.8	−4.5	−3.7	10.7	9.4	−4.5	−4.0	13.5	11.9	−4.5	−4.0	9.5	17.4

SECTIONS OF THE RAILS USED IN TESTS.

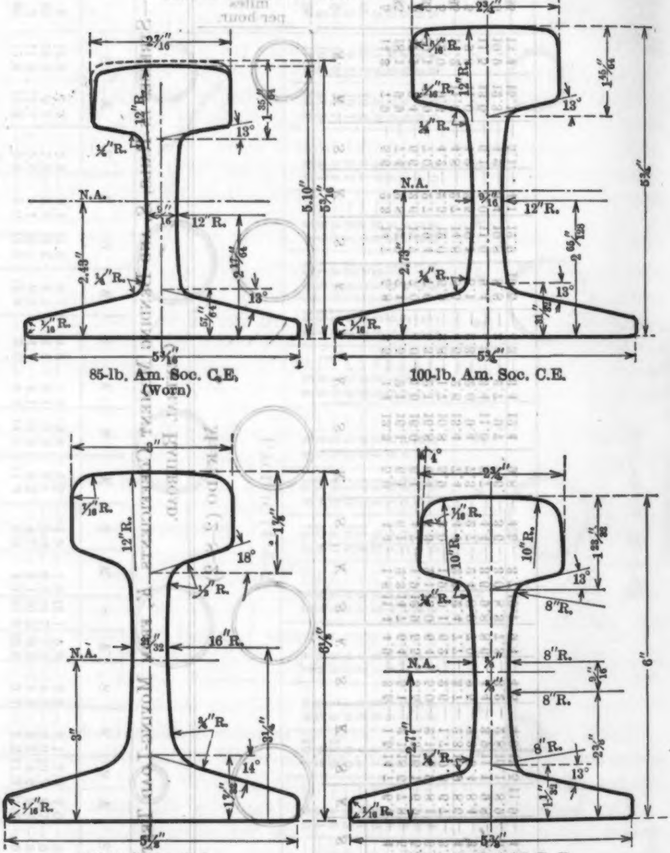


FIG. 107.

49.—Effect of Speed.—The position of the plotted points in Figs. 69 to 91 indicates that a rectilinear relation between stress in rail and speed of locomotive fits the data more generally and more satisfactorily than any other form. The variations of the plotted points from the straight lines of the diagrams are generally not greater than may be expected when variations in conditions of track and locomotive, errors of instruments and observations, and other accidental causes of variation are taken into account. Generally, for any test, the lines of a diagram are parallel, or nearly so.

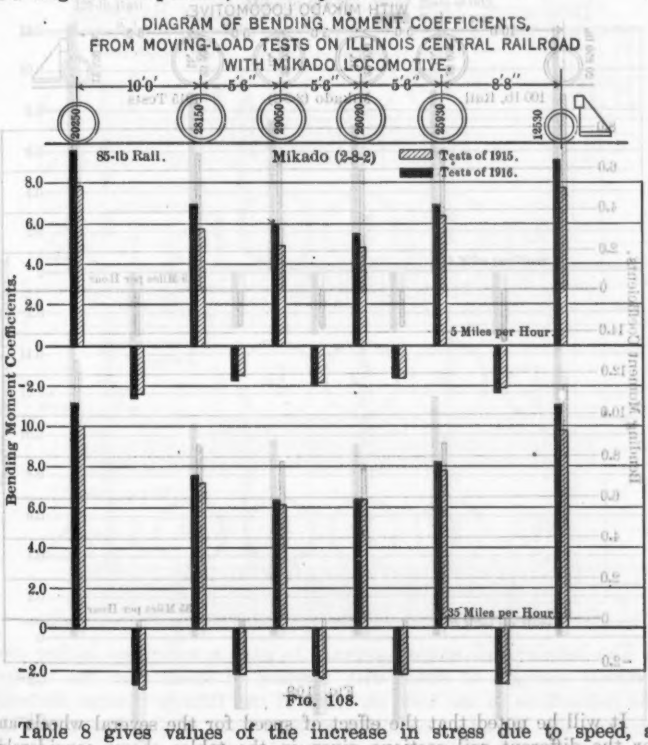


FIG. 108.

Table 8 gives values of the increase in stress due to speed, as obtained from the rectilinear relation of the diagrams. In the columns marked *A*, the increase is expressed as pound per square inch increase in stress for each mile per hour increase in speed greater than 5 miles per hour. In the columns marked *B*, the effect of speed is given as a percentage of the stress in the rail at 5 miles per hour for each mile per hour increase of speed greater than 5 miles per hour.

It will be seen that the values for the increase for positive moment range from about 0.3 to 1.2% increase for each mile per hour increase in speed. Values higher than 0.9% are found in a number of cases. The increases found in the tests on the Delaware, Lackawanna and Western Railroad, given in Table 9, are of the same character, but the values are somewhat smaller than those found in the tests on the Illinois Central Railroad.

DIAGRAM OF BENDING MOMENT COEFFICIENTS,
FROM MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.

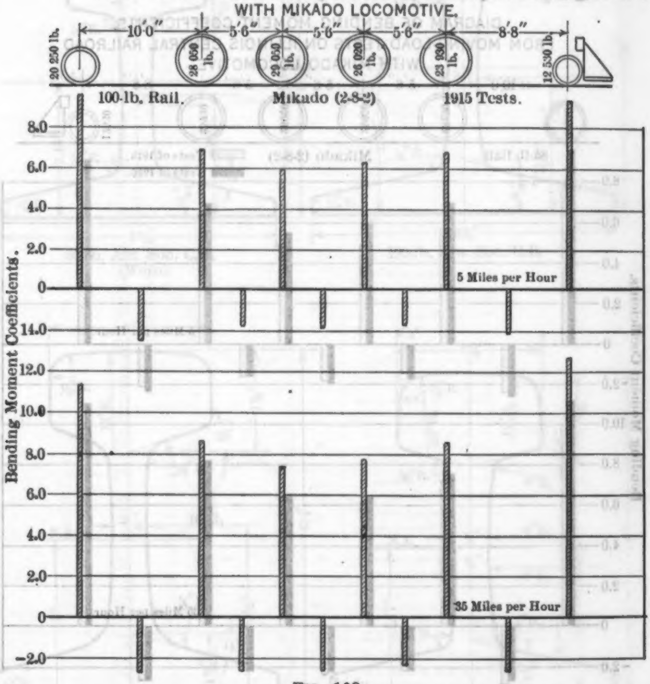


FIG. 109.

It will be noted that the effect of speed for the several wheels and for the different rail sections given in the tables shows considerable variation. The cause of these variations is not known. It may be due partly to the track, partly to the locomotives, and partly to accidental conditions of the runs of the tests. Although the runs were all made with the counterweight in its lowest position as it passed the middle instrument, some of the tests were made with the three

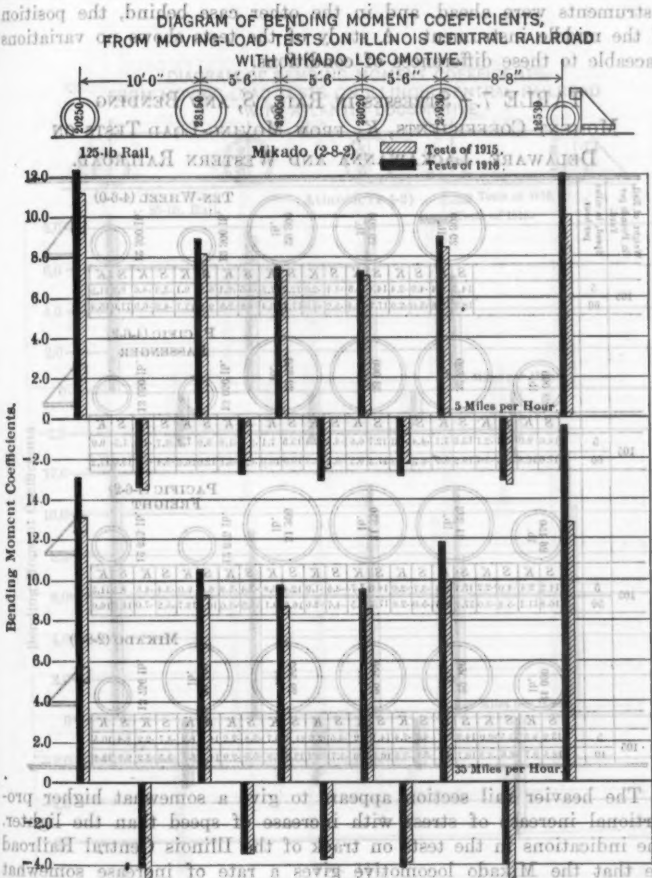


DIAGRAM OF BENDING MOMENT COEFFICIENTS,
FROM MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.
10'-0"
5'-6"
5'-6"
5'-6"
8'-8"
20250
28150
125-lb Rail
Mikado (2-8-2)
Tests of 1915.
Tests of 1916.
Bending Moment Coefficients.
12.0
10.0
8.0
6.0
4.0
2.0
0
-2.0
5 Miles per Hour.
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
-2.0
-4.0
35 Miles per Hour.

FIG. 110.

instruments on the rail at the right side of the locomotive and some with the three instruments at the left side; hence, in one case the counterweights on the other side of the locomotive opposite the three instruments were ahead, and in the other case behind, the position of the middle instrument. A study of the tests shows no variations traceable to these differences of conditions.

TABLE 7.—Stresses in Rail, S, and Bending Moment Coefficients, K, from Moving-Load Tests on Delaware, Lackawanna and Western Railroad.

Weight of Rail, in Pounds per Yard.	Speed, in miles per hour.																						
TEN-WHEEL (4-6-0)						29 300 lb.				29 300 lb.				29 300 lb.				12 300 lb.				12 300 lb.	
						S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K
105	5					14.3	8.4	-4.0	-2.4	14.4	8.5	-3.4	-2.0	12.7	7.5	-3.5	-2.1	6.5	9.1	-3.3	-4.6	8.2	11.5
	60					18.3	10.8	-5.0	-2.9	17.7	10.4	-5.2	-3.0	17.7	10.4	-4.8	-2.9	9.8	13.7	-4.9	-6.9	11.0	15.4
PACIFIC (4-6-2) PASSENGER		28 000 lb.				32 800 lb.				33 000 lb.				32 850 lb.				13 050 lb.				13 050 lb.	
		S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K
105	5	14.6	9.0	-5.1	-2.7	13.5	7.1	-4.4	-2.3	12.7	6.6	-4.5	-2.4	13.5	7.1	-3.4	-1.8	5.9	7.8	-3.7	-4.9	7.5	9.9
	60	17.6	10.8	-6.5	-3.5	19.2	9.6	-6.3	-3.3	17.3	9.1	-6.2	-3.2	19.0	10.0	-6.0	-3.1	12.6	16.6	-5.4	-8.5	13.0	17.2
PACIFIC (4-6-2) FREIGHT		26 150 lb.				31 333 lb.				31 330 lb.				31 300 lb.				12 675 lb.				12 675 lb.	
		S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K
105	5	14.2	9.4	-4.0	-2.2	15.1	8.3	-3.7	-2.0	14.0	7.7	-3.5	-1.9	12.4	6.8	-3.4	-1.9	6.6	9.0	-3.4	-4.6	8.5	11.5
	50	16.8	11.1	-5.4	-3.0	17.4	9.6	-5.0	-2.8	17.2	9.5	-4.7	-2.6	16.6	9.1	-5.5	-3.0	10.1	13.7	-5.2	-7.0	11.8	16.0
MIKADO (2-8-2)		21 000 lb.				29 560 lb.				29 560 lb.				29 560 lb.				29 560 lb.				13 750 lb.	
		S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K	S	K
105	5	13.2	9.5	-3.5	-2.0	14.9	8.1	-4.2	-2.4	14.1	8.2	-4.5	-2.6	11.6	6.7	-3.8	-2.2	14.1	8.2	-3.7	-2.2	5.4	10.5
	40	13.5	9.7	-6.6	-3.8	16.7	9.4	-5.6	-3.8	16.5	8.6	-5.7	-3.3	13.6	7.9	-5.0	-2.9	14.7	8.5	-5.5	-3.2	9.9	12.4

The heavier rail section appears to give a somewhat higher proportional increase of stress with increase of speed than the lighter. The indications in the tests on track of the Illinois Central Railroad are that the Mikado locomotive gives a rate of increase somewhat greater than the Atlantic and the Pacific. The tender trucks give a still higher rate of increase, though, of course, the amount of the stress is less than that under the drivers.

The proportional increase in the stress for negative moment is large and rather irregular, as would be expected from the smaller value of this stress and the greater variations of conditions to which

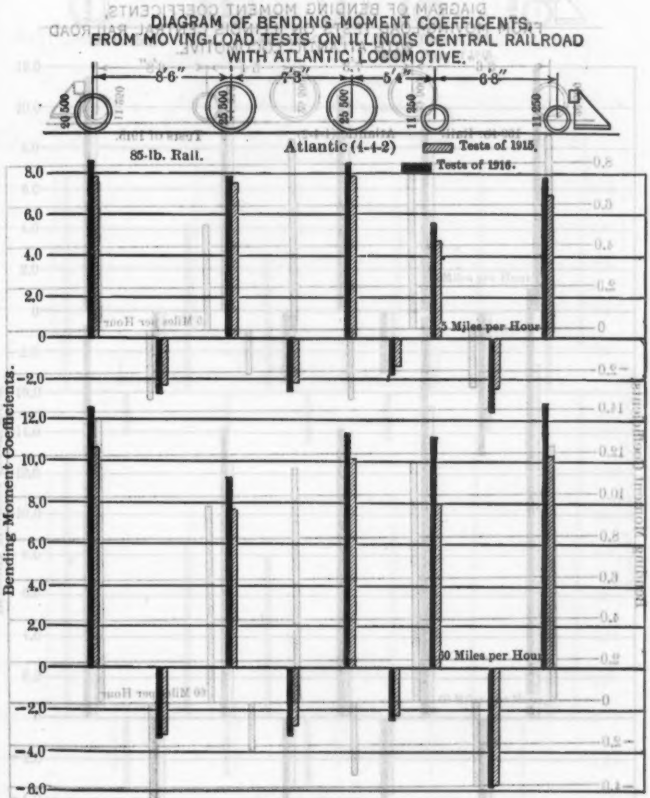


DIAGRAM OF BENDING MOMENT COEFFICENTS,
FROM MOVING LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH ATLANTIC LOCOMOTIVE.
8'6"
7'3"
5'4"
6'8"
20 500
25 500
25 500
11 250
11 250
85-lb. Rail.
Atlantic (4-4-2)
Tests of 1915.
Tests of 1916.
5 Miles per Hour
60 Miles per Hour
Bending Moment Coefficients.
8.0
6.0
4.0
2.0
0
−2.0
12.0
10.0
8.0
6.0
4.0
2.0
0
−2.0
−4.0
−6.0

FIG. 111.

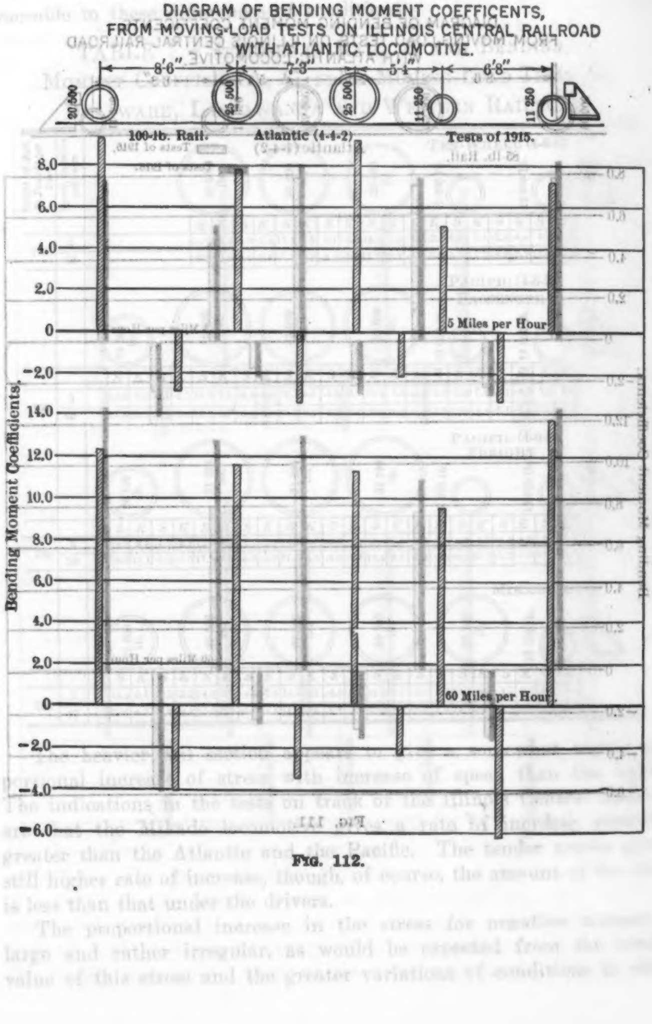


DIAGRAM OF BENDING MOMENT COEFFICENTS, FROM MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH ATLANTIC LOCOMOTIVE.
8'6"
7'3"
5'4"
6'8"
20 500
25 500
25 500
11 250
11 250
100-lb. Rail.
Atlantic (4-4-2)
Tests of 1915.
Bending Moment Coefficients.
8.0
6.0
4.0
2.0
0
−2.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
−2.0
−4.0
−6.0
5 Miles per Hour
60 Miles per Hour.

Fig. 112.

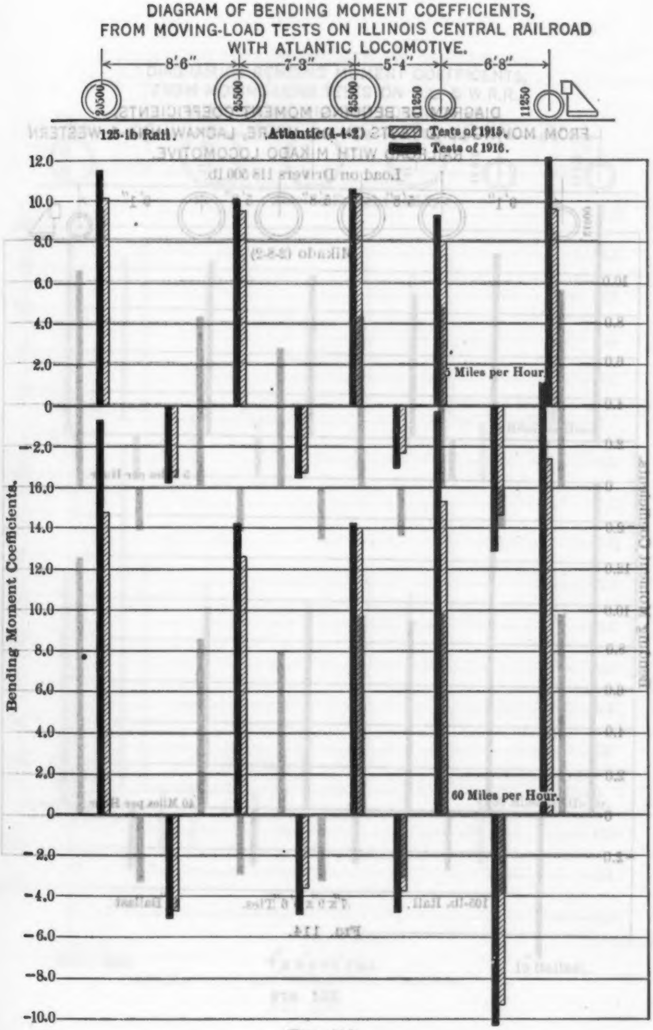


DIAGRAM OF BENDING MOMENT COEFFICIENTS,
FROM MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH ATLANTIC LOCOMOTIVE.
8'6"
7'3"
5'4"
6'8"
23500
25500
25500
11250
11250
125-lb Rail.
Atlantic (4-4-2)
Tests of 1915.
Tests of 1916.
Bending Moment Coefficients.
12.0
10.0
8.0
6.0
4.0
2.0
0
−2.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
−2.0
−4.0
−6.0
−8.0
−10.0
5 Miles per Hour.
60 Miles per Hour.

FIG. 113.

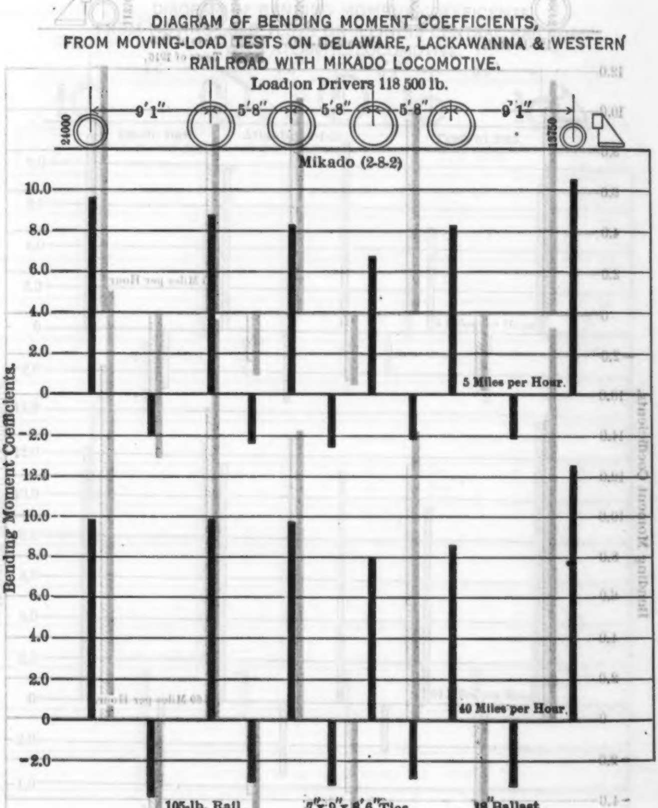


DIAGRAM OF BENDING MOMENT COEFFICIENTS,
FROM MOVING-LOAD TESTS ON DELAWARE, LACKAWANNA & WESTERN
RAILROAD WITH MIKADO LOCOMOTIVE.
Load on Drivers 118 500 lb.
9' 1"
5' 8"
5' 8"
5' 8"
9' 1"
24000
13750
Mikado (2-8-2)
Bending Moment Coefficients.
10.0
8.0
6.0
4.0
2.0
0
-2.0
12.0
5 Miles per Hour.
40 Miles per Hour.
105-lb. Rail.
7"x 9"x 8' 6" Ties.
18" Ballast.

FIG. 114.

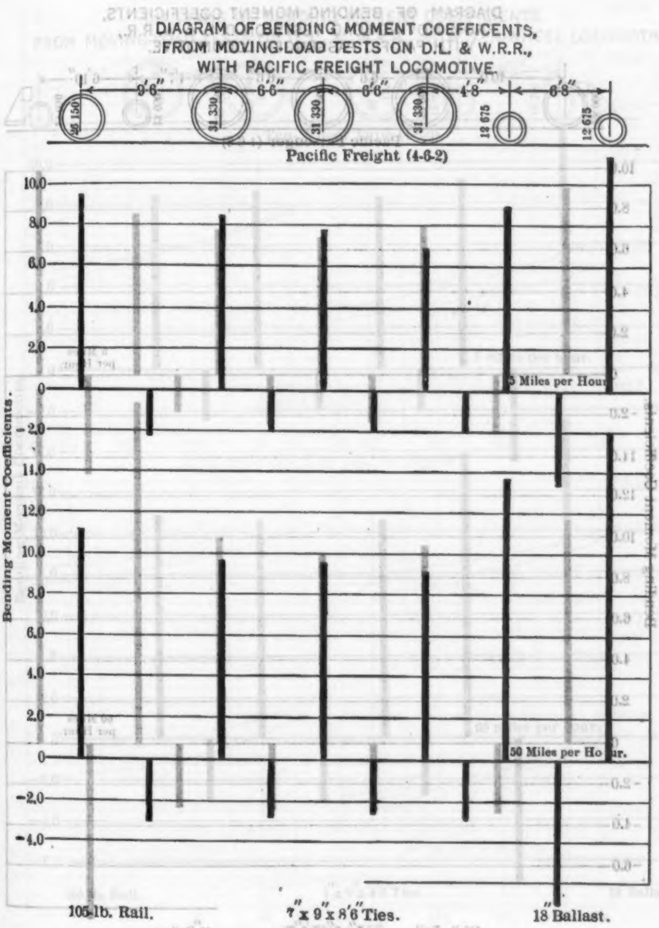


DIAGRAM OF BENDING MOMENT COEFFICENTS,
FROM MOVING-LOAD TESTS ON D.L. & W.R.R.,
WITH PACIFIC FREIGHT LOCOMOTIVE.
9'6"
6'6"
6'6"
4'8"
6'8"
26 150
31 330
31 330
31 330
12 675
12 675
Pacific Freight (4-6-2)
Bending Moment Coefficients.
10.0
8.0
6.0
4.0
2.0
0
-2.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
-2.0
-4.0
5 Miles per Hour
50 Miles per Hour.
105-lb. Rail.
7" x 9" x 8'6" Ties.
18" Ballast.

Fig. 115.

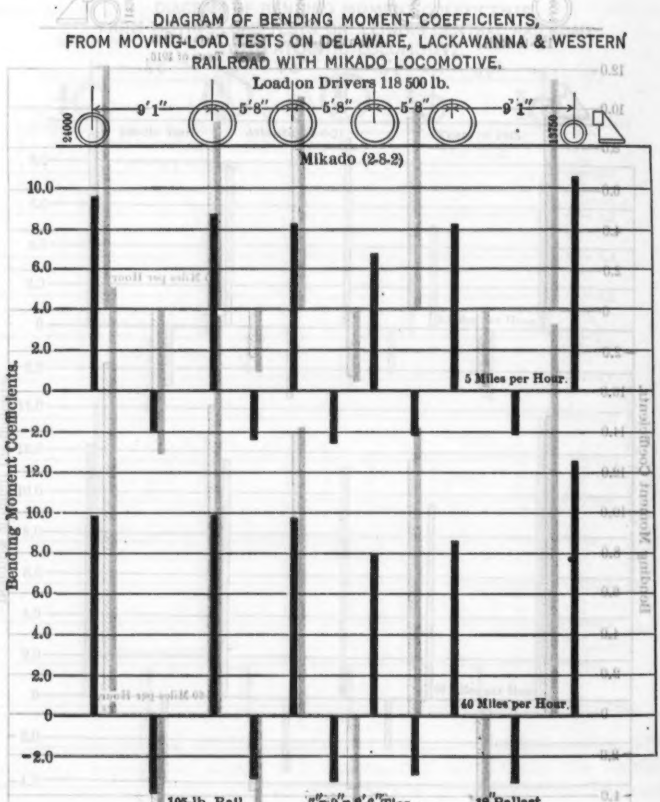


DIAGRAM OF BENDING MOMENT COEFFICIENTS,
FROM MOVING-LOAD TESTS ON DELAWARE, LACKAWANNA & WESTERN RAILROAD WITH MIKADO LOCOMOTIVE.
Load on Drivers 118 500 lb.
9′1″
5′8″
5′8″
5′8″
9′1″
24000
13750
Mikado (2-8-2)
Bending Moment Coefficients.
10.0
8.0
6.0
4.0
2.0
0
-2.0
12.0
5 Miles per Hour.
40 Miles per Hour.
105-lb. Rail.
7″x 9″x 8′6″ Ties.
18″ Ballast.

Fig. 114.

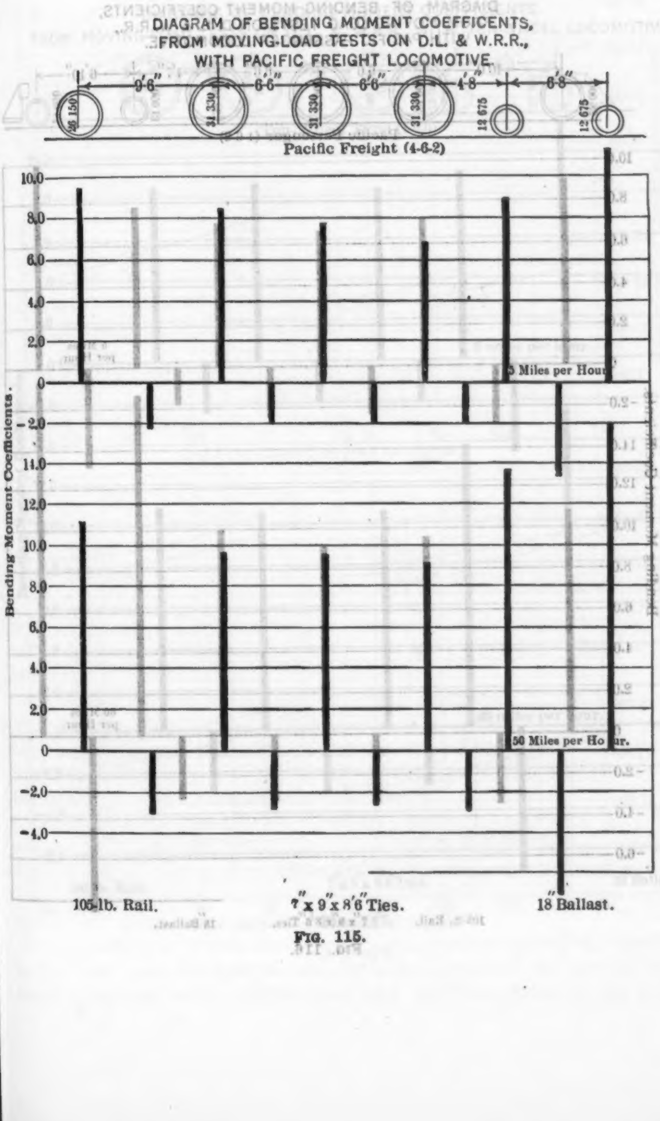


DIAGRAM OF BENDING MOMENT COEFFICENTS,
FROM MOVING-LOAD TESTS ON D.L. & W.R.R.,
WITH PACIFIC FREIGHT LOCOMOTIVE.
9′6″
6′6″
6′6″
4′8″
6′8″
26 150
31 330
31 330
31 330
12 675
12 675
Pacific Freight (4-6-2)
Bending Moment Coefficients.
5 Miles per Hour
50 Miles per Hour.
105-lb. Rail.
7″ x 9″ x 8′6″ Ties.
18″ Ballast.

FIG. 115.

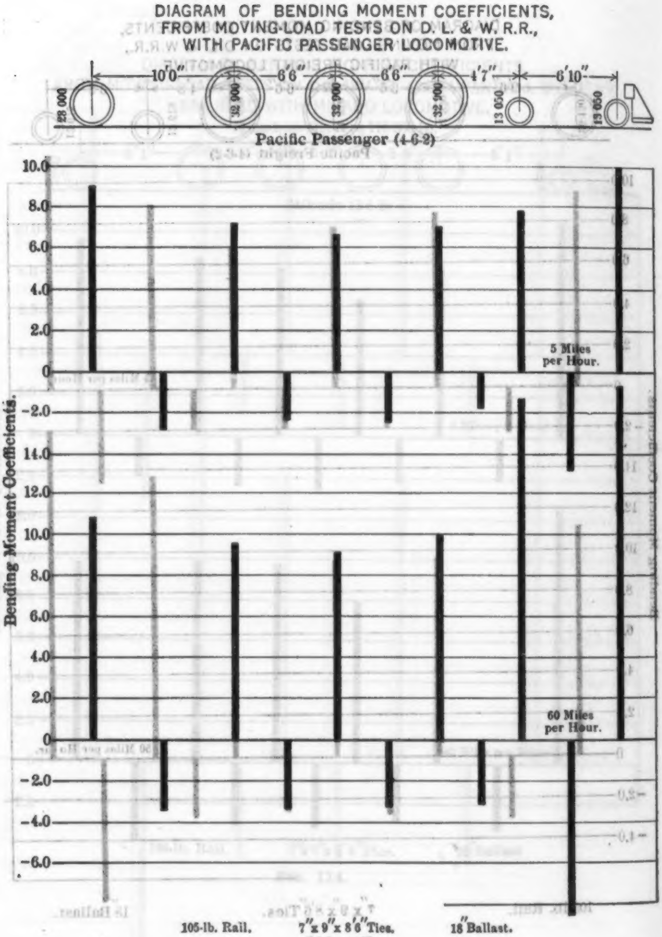


DIAGRAM OF BENDING MOMENT COEFFICIENTS, FROM MOVING-LOAD TESTS ON D. L. & W. R.R., WITH PACIFIC PASSENGER LOCOMOTIVE.
10'0"
6'6"
6'6"
4'7"
6'10"
28 000
32 900
32 900
32 900
13 050
13 050
Pacific Passenger (4-6-2)
Bending Moment Coefficients.
5 Miles per Hour.
60 Miles per Hour.
105-lb. Rail.
7"x 9"x 8'6" Ties.
18" Ballast.

FIG. 116.

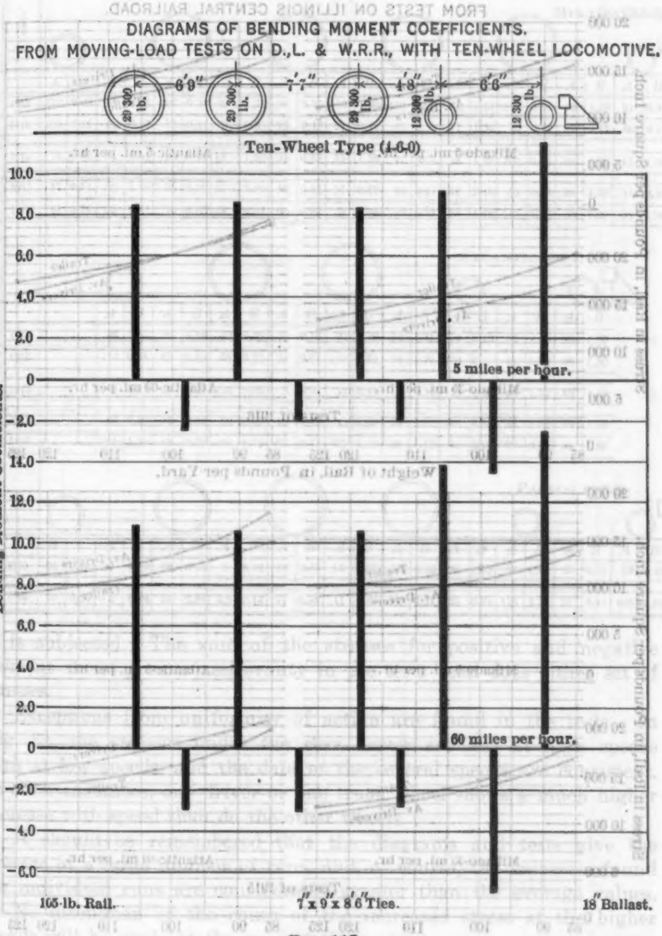

DIAGRAMS OF BENDING MOMENT COEFFICIENTS.
FROM MOVING-LOAD TESTS ON D.,L. & W.R.R., WITH TEN-WHEEL LOCOMOTIVE.
29 300 lb.
6'9"
29 300 lb.
7'7"
29 300 lb.
4'8"
12 300 lb.
6'6"
12 300 lb.
Ten-Wheel Type (4-6-0)
5 miles per hour.
60 miles per hour.
Bending Moment Coefficients.
10.0
8.0
6.0
4.0
2.0
0
-2.0
14.0
12.0
-4.0
-6.0
105-lb. Rail.
7"x 9" x 8'6" Ties.
18" Ballast.

FIG. 117.

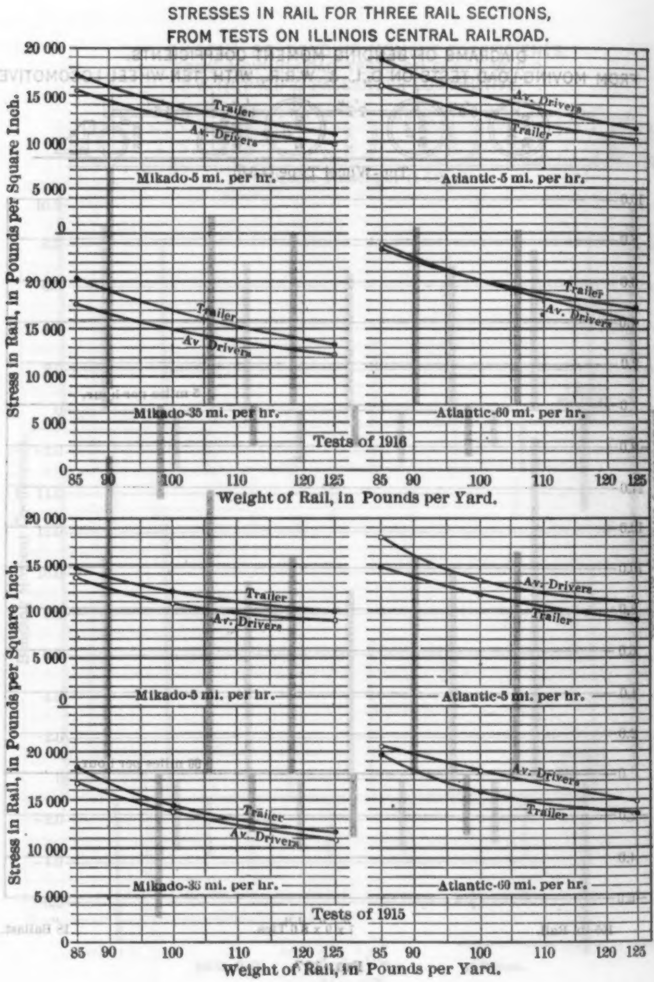

STRESSES IN RAIL FOR THREE RAIL SECTIONS,
FROM TESTS ON ILLINOIS CENTRAL RAILROAD.
Stress in Rail, in Pounds per Square Inch.
Trailer
Av. Drivers
Mikado-5 mi. per hr.
Atlantic-5 mi. per hr.
Mikado-35 mi. per hr.
Atlantic-60 mi. per hr.
Tests of 1916
Tests of 1915
Weight of Rail, in Pounds per Yard.
20 000
15 000
10 000
5 000
0
85
90
100
110
120
125

FIG. 118.

TABLE 8.—INCREASE IN STRESS IN RAIL DUE TO SPEED. TESTS ON ILLINOIS CENTRAL RAILROAD.

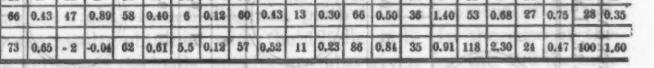
MIKADO (2-8-2)

Weight of Rail, in Pounds per Yard.	Year.	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
85	1915	132	0.93	53	0.98	120	0.82	56	1.73	96	0.77	53	1.30	120	1.10	.53	1.60	107	0.74	60	1.28	81	0.95
85	1916	108	0.64	24	0.45	78	0.44	3.9	0.95	29	0.18	21	0.42	81	0.59	41	1.30	72	0.44	39	0.77	80	0.73
100	1915	75	0.63	21	0.51	93	0.78	48	1.90	94	0.88	45	1.50	77	0.77	38	1.50	102	0.95	38	1.20	85	1.20
125	1915	57	0.51	62	1.50	50	0.49	67	2.90	55	0.58	57	2.10	62	0.77	62	2.40	55	0.58	59	1.70	54	0.98
125	1916	85	0.78	32	0.87	70	0.65	28	0.83	90	0.92	30	1.00	78	0.94	43	1.40	115	1.20	35	1.20	95	1.50

ATLANTIC (4-4-2)

Weight of Rail, in Pounds per Yard.	Year.	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
85	1915	96	0.65	30	0.59	53	0.29	32	0.76	53	0.30	40	1.40	94	2.10	63	1.30	93	1.30
85	1916	133	0.84	25	0.45	59	0.33	32	0.65	119	0.61	34	0.91	104	2	41	1.20	95	1.20
100	1915	71	0.60	40	0.93	116	0.86	5	0.11	58	0.40	9	0.32	55	1.50	40	1.80	82	1.60
125	1915	82	0.90	31	0.84	62	0.59	13	0.42	73	0.63	31	1.20	66	1.70	38	1.50	69	1.50
125	1916	124	1.20	28	0.71	82	0.72	27	0.71	75	0.63	35	1.10	95	2.10	29	0.88	80	1.30

PACIFIC (4-6-2)



Weight of Rail, in Pounds per Yard.	Year.	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
85	1915	66	0.43	47	0.89	58	0.40	6	0.12	60	0.43	13	0.30	66	0.50	36	1.40	53	0.68	27	0.75	28	0.35
125	1916	73	0.65	-2	-0.04	62	0.61	5.5	0.12	57	0.52	11	0.23	86	0.84	35	0.91	118	2.30	24	0.47	100	1.60

it is subjected. The sum of the stresses for positive and negative moment shows greater uniformity in the tests than does either set of stresses.

Exceptions from uniformity of action are found in the tests. In Fig. 82, the stresses under the rear driver are less at high speeds than at low speeds, and the data at the several speeds are consistent. In other instances, one driver or one truck wheel shows a much higher increase with speed than do the other wheels.

It should be remembered that the diagrams and tests give the average of a large number of runs, and, of course, the increases found for individual runs are considerably greater than the average values.

No discussion of the cause of the increased stress at the higher speeds will be made at the present time.

50.—Influence of Rail Section.—In structural members, the bending moment for a given load is usually independent of the section of a beam or girder which carries load, and the fiber stress in the beam

TABLE 9.—Increase in Stress in Rail Due to Speed. Tests on Delaware, Lackawanna and Western Railroad.

Ten-Wheel (4-6-0)

Weight of Rail, in Pounds per Yard.	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
105	73	0.51	18	0.45	60	0.42	33	0.97	91	0.72	24	0.67	60	0.92	29	0.88	51	0.62

Pacific (4-6-2)

Weight of Rail, in Pounds per Yard.	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	
105	55	0.37	25	0.50	86	0.63	35	0.79	84	0.66	31	0.69	98	0.72	47	1.39	122	2.06	49	1.33	100	1.33	Pass.
105	58	0.41	31	0.78	51	0.34	29	0.78	71	0.51	27	0.76	93	0.75	47	1.37	78	1.18	40	1.18	73	0.86	Frt.

Mikado (2-8-2)

Weight of Rail, in Pounds per Yard.	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
105	9	0.06	89	2.53	52	0.35	40	0.95	69	0.49	34	0.76	57	0.49	34	0.90	17	1.21	52	1.39	43	0.51

is inversely proportional to the section modulus, $\frac{I}{c}$. For the track structure, it has been shown by analysis that, for a one-axle load, the bending moment developed varies with the rail section, becoming larger as the moment of inertia of the section is increased; and, therefore, in calculating the fiber stress produced by a given load in a given rail section, the value of the bending moment developed with the use of that rail section must be known. For a two-axle or a three-axle load, or for a combination of driver loads and truck wheel loads, the magnitude of the bending moment will still be dependent on the rail section, but not to the same extent as with a single load; and the influence of the rail section on the bending moment developed will also depend on the wheel spacing. The general analytical relations involving the effect of the rail section on the bending moment have already been derived. Values found from the tests will now be presented. It will be most convenient to use in the discussion the bending moment coefficient, K, by which the wheel load, P, may be multiplied to get the bending moment at the load or at a point between loads.

The results are presented graphically, the values of the bending moment coefficients obtained at the drivers and other loads being plotted against the moment of inertia of the rail section. Fig. 119 gives the values from the tests on the Illinois Central Railroad for one-axle and two-axle loads applied statically and for three types of locomotive at two speeds. The values at the left of each diagram, plotted for a moment of inertia of 27, are results obtained with the worn 85-lb. rail (see Table 5 for properties of the rail sections). The values at the right of each diagram, plotted for a moment of inertia of 68.7, are results obtained with the 125-lb. rail. In the case of the one-axle and two-axle loads, results are also given for the 100-lb. rail, the moment of inertia of the section being 44. The moving-load tests with the Mikado and Atlantic locomotives on the 100-lb. rail were made in 1915, and, therefore, with the old form of stremmatograph; the tests with the 85-lb. and 125-lb. rails, the results of which are plotted on Fig. 119, were made in 1916 with the new form of stremmatograph. For this reason, the tests with the Mikado and Atlantic locomotives on the 100-lb. rail are not plotted. Only 1915 tests are available for the Pacific locomotive and the 85-lb. rail; in comparing with the bending moment coefficients for the 125-lb. rail from the 1916 tests, it should be kept in mind that the 1915 values are probably somewhat low. For convenience in comparing results, lines have been drawn connecting corresponding points.

The upper left diagram of Fig. 119 gives values of the bending moment coefficient for the one-axle load tests with the three rail sections. In the static-load tests as readings were taken on only one side of the rail, except in the 1917 tests, the values may not be directly comparable, though, for this form of test, not much variation has

been found for the two sides of the rail. It is seen that the heavier rail gives a much higher bending moment coefficient than the lighter. If the difference in the track conditions is taken into account, the 125-lb. rail being on track with 24-in. ballast, which gave a higher modulus of elasticity of rail-support than that of the track on which the tests of 85-lb. rail here used were made, the contrast becomes greater. Fig. 119 also gives the analytical values of the bending moment coefficients for a one-axle load based on the probable values of the modulus of elasticity of rail-support used in the tests, 1 000 in one case and 1 600 in the other. The trend of the analytical values is much the same as that derived from the test data.

The upper right diagram of Fig. 119 gives values of the bending moment coefficient corresponding to the observed stress found in the three rail sections for the two-axle load tests. The wheel spacing is 66 in. The analytical values of the bending moment coefficients are also given. The same values of the modulus of elasticity of rail-support are used as for the one-axle load tests. The trend of the values shows higher bending moment coefficients for the heavier rail sections.

Fig. 119 also gives values of the bending moment coefficients corresponding to the stresses observed in the moving-load tests with the Mikado, Atlantic, and Pacific locomotives. Values are given for speeds of 5 and 35 or 60 miles per hour. The bending moment coefficients for the heavier rail sections are markedly larger than for the lighter sections, and this is true for the drivers and the trailer. The values of the coefficients of negative bending moment are not plotted here, but these also show an increased coefficient for the heavier rail sections. That the bending moment coefficients should be greater for the heavier sections is evident from analytical considerations, especially in the case of the trailer, which is at some distance from other wheels, and in the case of outer drivers. The increase due to increase of section, however, is greater than may be expected from the analysis of track action herein given, or for any known rational analysis. For the drivers and trailer of the three types of locomotives used on the Illinois Central Railroad, the values of the bending moment coefficient derived by the analysis are less than 10% higher for track with the 125-lb. than for track with the 85-lb. section. For the tests at 5 miles per hour, the values of the bending moment coefficients average nearly 30% higher with the 125-lb. than with the 85-lb. section, considering the drivers and the trailer of the three types of locomotive. For the tests at the higher speeds, the increase is still greater.

On Fig. 119 also are plotted points for the bending moment coefficients derived from the tests on the Delaware, Lackawanna and Western Railroad with the Mikado and Pacific locomotives, the rail being 105-lb. section. The wheel spacing and wheel loads in these locomotives are, of course, not the same as those of the same type on the Illinois Central Railroad.

BENDING MOMENT COEFFICIENTS FOR THREE RAIL SECTIONS.

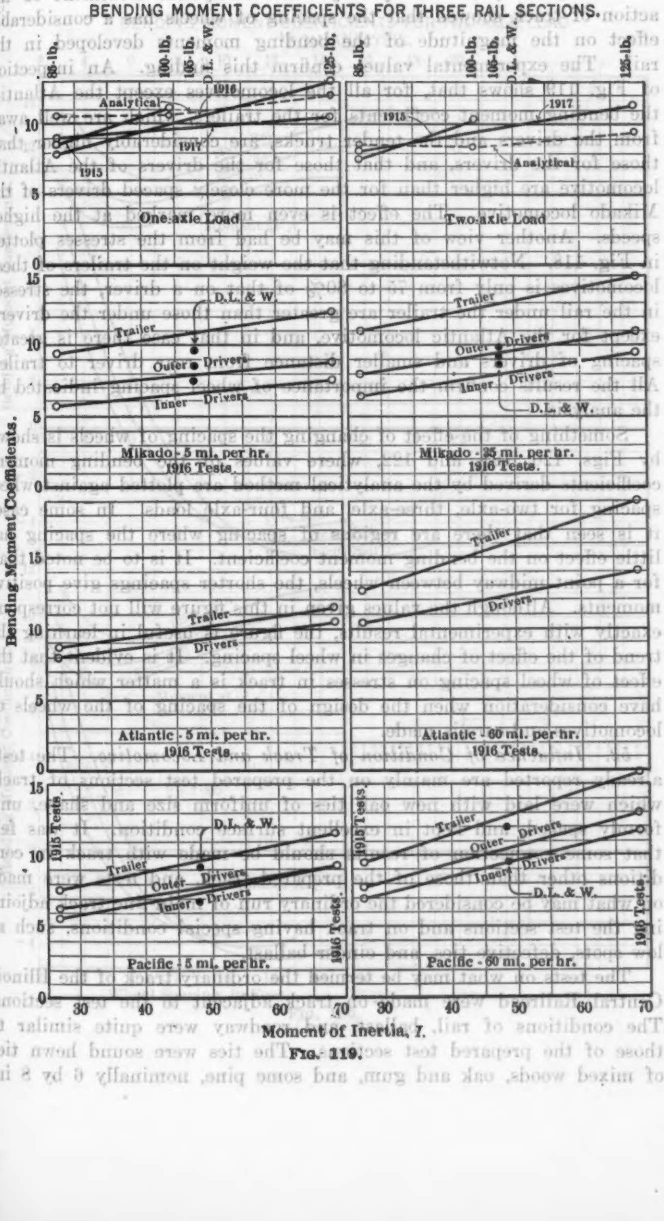


FIG. 119.

51.—Effect of Wheel Spacing.—The analytical treatment of the action of track showed that the spacing of wheels has a considerable effect on the magnitude of the bending moments developed in the rail. The experimental values confirm this finding. An inspection of Fig. 119 shows that, for all the locomotives except the Atlantic. the bending moment coefficients for the trailers, which are well away from the drivers and the tender trucks, are considerably higher than those for the drivers, and that those for the drivers of the Atlantic locomotive are higher than for the more closely spaced drivers of the Mikado locomotive. The effect is even more marked at the higher speeds. Another view of this may be had from the stresses plotted in Fig. 118. Notwithstanding that the weight on the trailers of these locomotives is only from 75 to 80% of that on a driver, the stresses in the rail under the trailer are greater than those under the drivers, except for the Atlantic locomotive, and in that case there is greater spacing of drivers and smaller distance from rear driver to trailer. All the results confirm the importance of wheel spacing indicated by the analysis.

Something of the effect of changing the spacing of wheels is shown by Figs. 120, 121, and 122, where values of the bending moment coefficients derived by the analytical method are plotted against wheel spacing for two-axle, three-axle, and four-axle loads. In some cases it is seen that there are regions of spacing where the spacing has little effect on the bending moment coefficient. It is to be noted that, for a point midway between wheels, the shorter spacings give positive moments. Although the values given in this figure will not correspond exactly with experimental results, the figure is useful in learning the trend of the effect of changes in wheel spacing. It is evident that the effect of wheel spacing on stresses in track is a matter which should have consideration when the design of the spacing of the wheels of locomotives and cars is made.

52.—Influence of Condition of Track and Locomotive.—The tests already reported are mainly on the prepared test sections of track, which were laid with new oak ties of uniform size and shape, uniformly spaced, and kept in excellent surface condition. It was felt that some connection of results should be made with track in conditions other than those of the prepared track, and tests were made on what may be considered the ordinary run of main-line track adjoining the test sections and on track having special conditions, such as low spots, defective ties, and cinder ballast.

The tests on what may be termed the ordinary track of the Illinois Central Railroad were made on track adjacent to the test sections. The conditions of rail, ballast, and roadway were quite similar to those of the prepared test sections. The ties were sound hewn ties of mixed woods, oak and gum, and some pine, nominally 6 by 8 in.

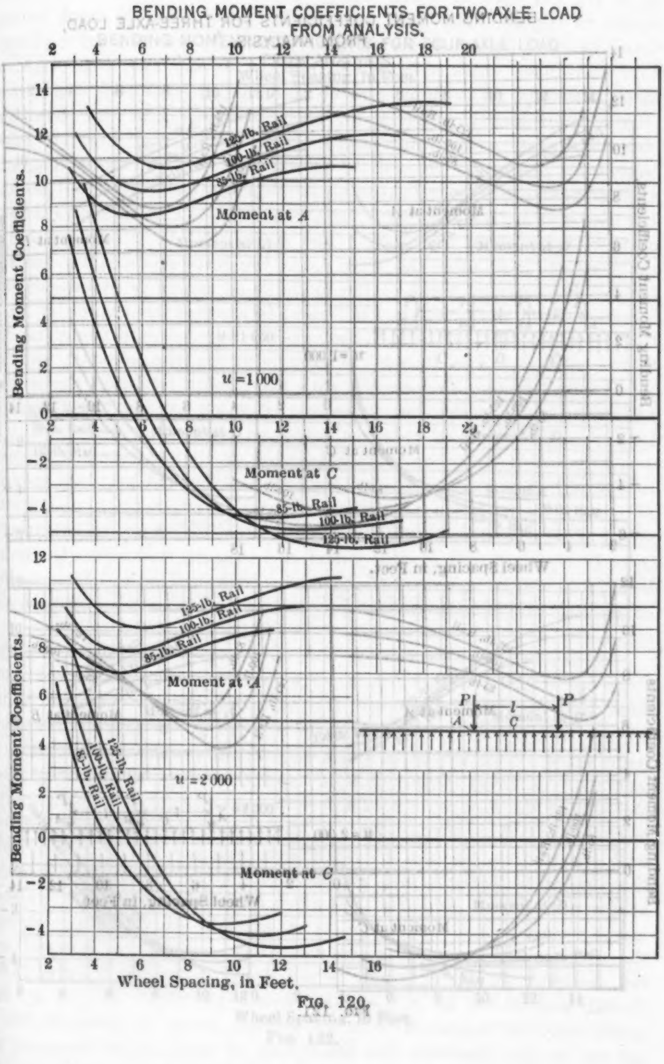

BENDING MOMENT COEFFICIENTS FOR TWO-AXLE LOAD FROM ANALYSIS.
Bending Moment Coefficients.
125-lb. Rail
100-lb. Rail
85-lb. Rail
Moment at A
u = 1000
Moment at C
u = 2000
P
l
P
A
C
Wheel Spacing, in Feet.

FIG. 120.

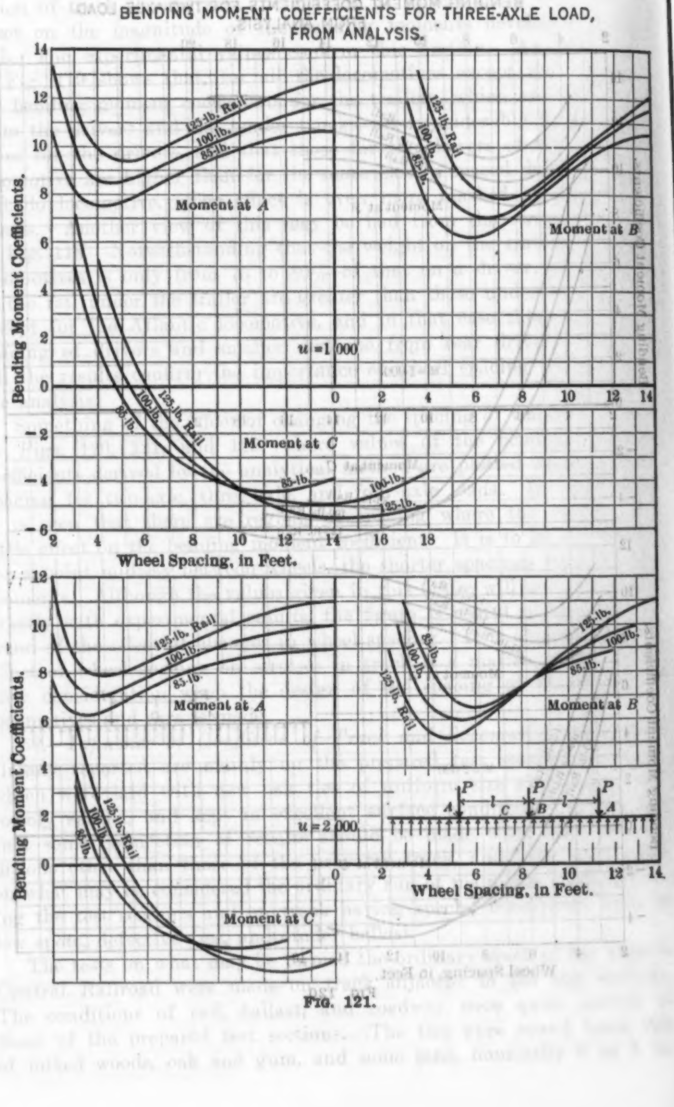

BENDING MOMENT COEFFICIENTS FOR THREE-AXLE LOAD, FROM ANALYSIS.
Bending Moment Coefficients.
125-lb. Rail
100-lb.
85-lb.
Moment at A
Moment at B
u = 1 000
Moment at C
Wheel Spacing, in Feet.
u = 2 000
P
l
A
B
C
Wheel Spacing, in Feet.

FIG. 121.

BENDING MOMENT COEFFICIENTS FOR FOUR-AXLE LOAD FROM ANALYSIS.

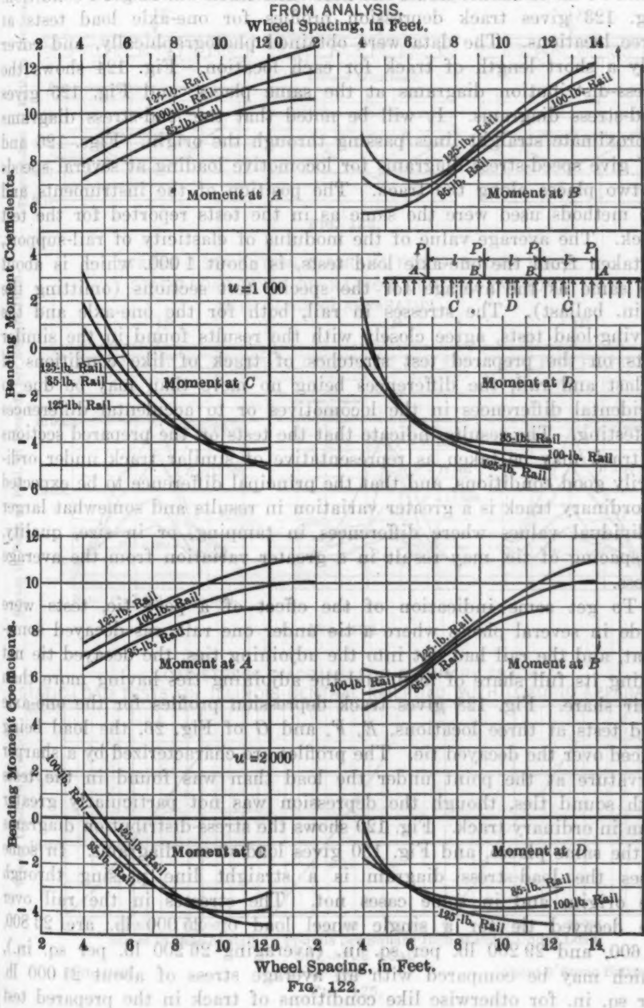


FIG. 122.

by 8 ft., the spacing being fairly uniform, and about 20 in. from center to center. The tests were made on stretches of track having 6 and 24 in. of ballast under the ties. The track was in good condition. Fig. 123 gives track depression profiles for one-axle load tests at three locations. The data were obtained photographically, and cover only a short length of track for each location. Fig. 124 shows the stress-distribution diagrams at the same places, and Fig. 125 gives load-stress diagrams. It will be noted that the load-stress diagrams approximate straight lines passing through the origin. Figs. 126 and 127 give speed-stress diagrams for locomotive loading at several speeds at two places along the track. The position of the instruments and the methods used were the same as in the tests reported for the test track. The average value of the modulus of elasticity of rail-support, *u*, taken from the one-axle load tests, is about 1 000, which is about the same as the average for the special test sections (omitting the 24-in. ballast). The stresses in rail, both for the one-axle and the moving-load tests, agree closely with the results found in the similar tests on the prepared test stretches of track of like conditions of ballast and rail, the differences being no more than may be due to accidental differences in the locomotives or to accidental differences in testing. The results indicate that the tests on the prepared sections of track may be taken as representative of similar track under ordinarily good conditions, and that the principal difference to be expected in ordinary track is a greater variation in results and somewhat larger individual values where differences in tamping, or in size, quality, or spacing of tie, may result in a greater variation from the average stress.

To get some indication of the effect of a poor tie, tests were made in several places where a tie under one rail was decayed somewhat, and the rail had cut into the adjoining ties, the decayed tie not taking its full share of load, and the adjoining ties having more than their share. Fig. 128 gives track depression profiles for the one-axle load tests at three locations, *E*, *F*, and *G* of Fig. 26, the load being placed over the decayed tie. The profiles are characterized by a sharper curvature at the point under the load than was found in the tests with sound ties, though the depression was not particularly greater than in ordinary track. Fig. 129 shows the stress-distribution diagrams at the same places, and Fig. 130 gives load-stress diagrams. In some cases the load-stress diagram is a straight line passing through the origin, and in some cases not. The stresses in the rail over the decayed tie for a single wheel load of 25 000 lb. are 23 800, 25 600, and 29 200 lb. per sq. in. (averaging 26 200 lb. per sq. in.), which may be compared with an average stress of about 21 000 lb. per sq. in. for otherwise like conditions of track in the prepared test sections. Moving-load tests with the Mikado locomotive were also

TRACK DEPRESSION PROFILES FOR ORDINARY TRACK.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD,
WITH LOADING APPARATUS.

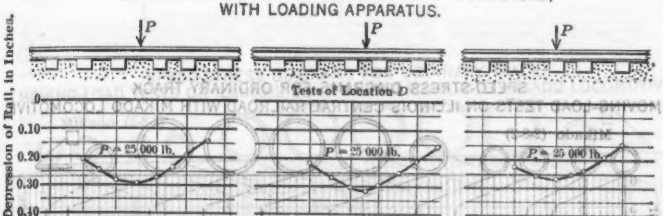


FIG. 123.

STRESS DISTRIBUTION DIAGRAMS FOR ORDINARY TRACK.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD,
WITH LOADING APPARATUS.

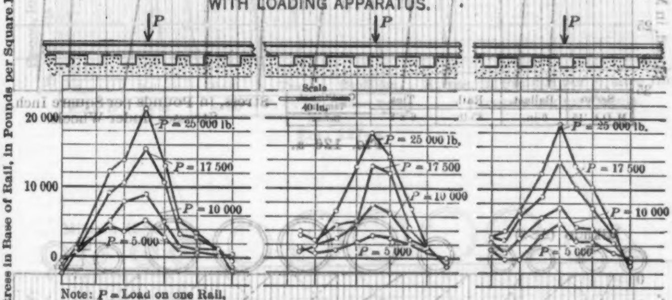
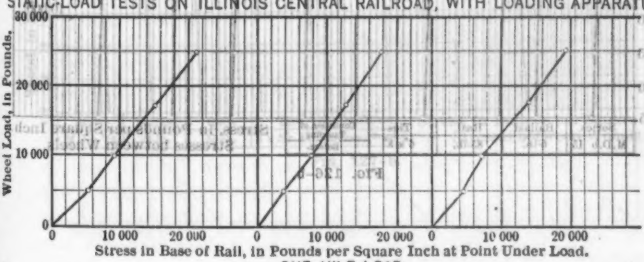


FIG. 124.

LOAD-STRESS DIAGRAMS FOR ORDINARY TRACK.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD, WITH LOADING APPARATUS.



ONE-AXLE LOAD

85-lb. Rail. 6" x 8" x 8' Ties About 6" Stone Ballast.

FIG. 125.

SPEED-STRESS DIAGRAMS FOR ORDINARY TRACK
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.

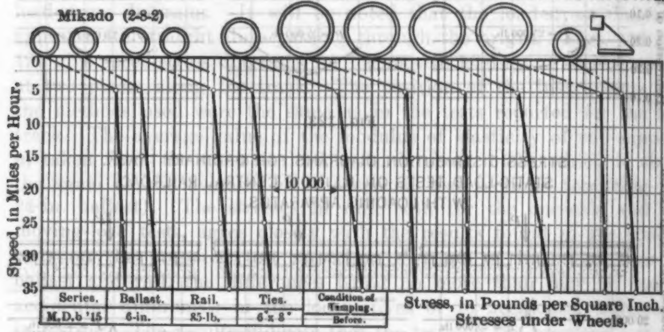


FIG. 126–a.

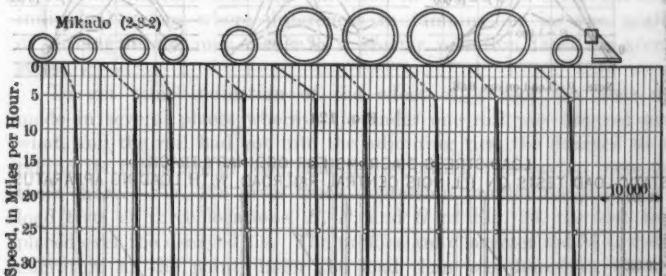


FIG. 126–b.

SPEED-STRESS DIAGRAMS FOR ORDINARY TRACK

MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.

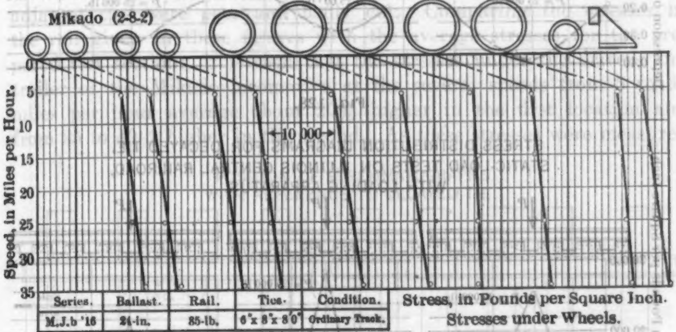


FIG. 127–a.

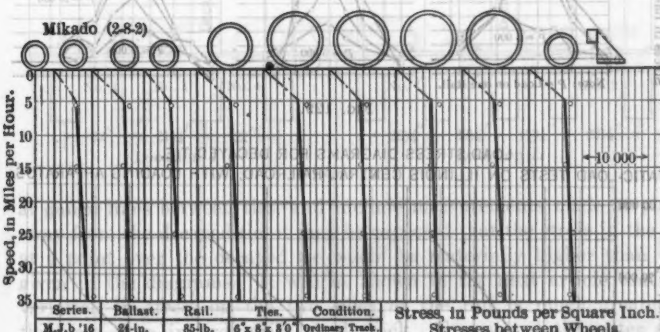


FIG. 127–b.

TRACK DEPRESSION PROFILES FOR DECAYED TIE.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD, WITH LOADING APPARATUS.

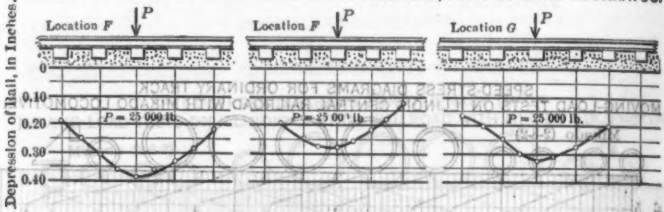


FIG. 128.

STRESS DISTRIBUTION DIAGRAMS FOR DECAYED TIE.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD,
WITH LOADING APPARATUS.

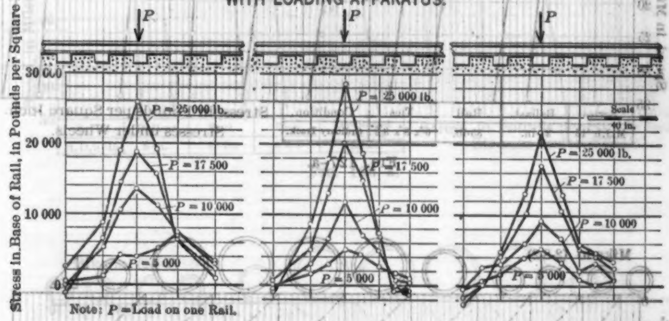
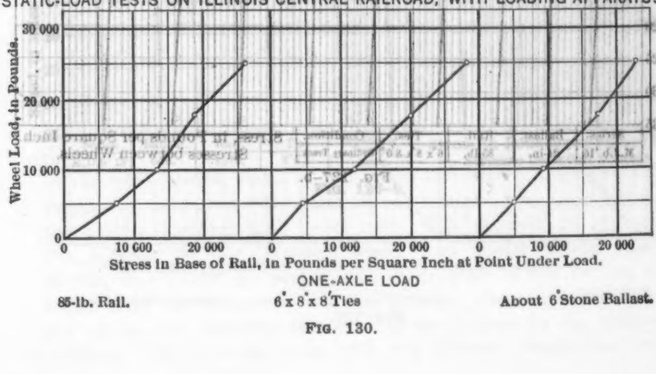


FIG. 129.

LOAD-STRESS DIAGRAMS FOR DECAYED TIE.
STATIC-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD, WITH LOADING APPARATUS.



ONE-AXLE LOAD

85-lb. Rail. 6" x 8" x 8' Ties About 6" Stone Ballast.

FIG. 130.

made at the two locations, *I* and *L*, shown on Fig. 26. Fig. 131 gives the position of the instruments with reference to the decayed tie, and Figs. 132 and 133 are speed-stress diagrams giving the average of the stresses obtained with three instruments. At *I*, the decayed tie was in such condition that it gave little bearing resistance to the rail load, and the two adjoining ties were sound, but somewhat cut by the rail. At *L*, the decayed tie offered some bearing resistance, but the adjacent ties were poor and badly cut. Comparing the stresses in the rail given in these figures with the average stresses for the prepared test sections, it is seen that the stresses under the drivers and trailer of the Mikado locomotive at the several speeds from 5 to 35 miles per hour average about 20% higher in the first location and from 35 to 40% higher in the second. As these stresses were measured

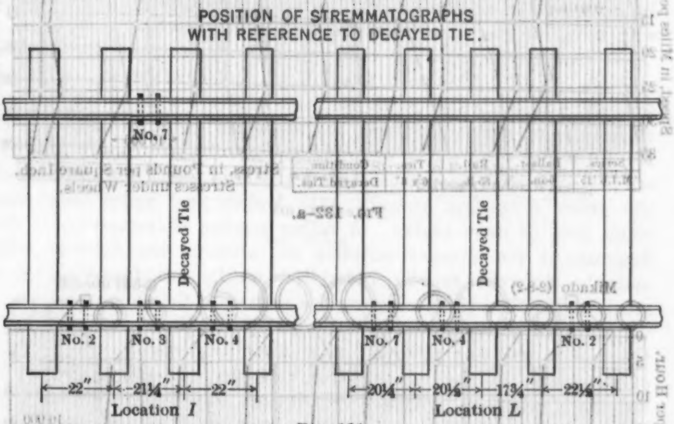


FIG. 131.

at gauge lines between ties, and include measurements made at places away from the decayed tie, it may be expected that the stress in the rail at a point over the decayed tie would be materially higher than that given in the diagrams. It will be shown in the next article that the variation in stress on the two sides of the rail is much greater in this case than in ordinarily good track. Stresses at the outer edge of base of rail were found which average about 150% higher than the average rail stress for good track.

Tests were made on a spot where, for about a rail length, the track was low, especially along one rail. A low spot on the other rail, a rail length or so away, served to give the locomotive a roll at speeds greater than 5 miles per hour. The sags were quite apparent to the eye. The section foreman explained the presence of the low spots as caused by

SPEED-STRESS DIAGRAMS FOR DECAYED TIE.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.

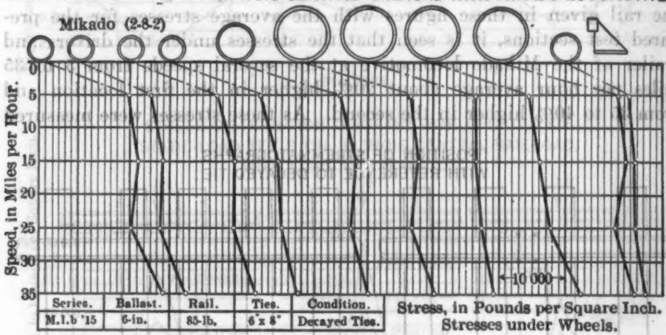


FIG. 132–a.

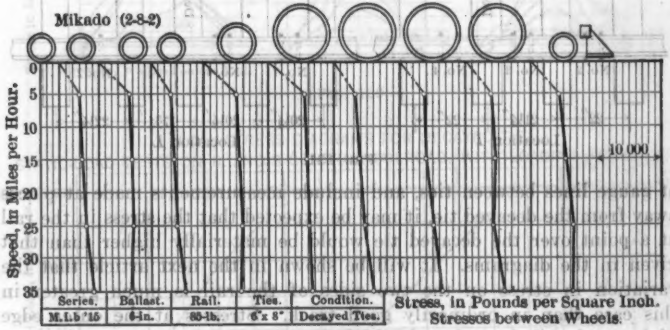


FIG. 132–b.

SPEED-STRESS DIAGRAMS FOR DECAYED TIE.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD WITH MIKADO LOCOMOTIVE.

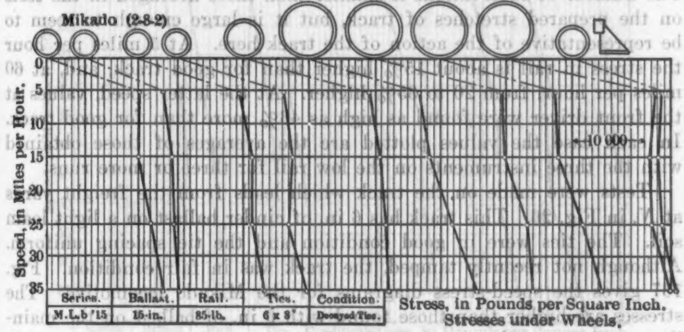


FIG. 133–a.

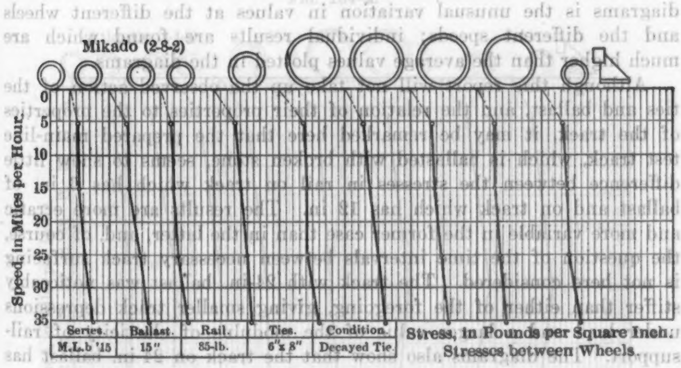


FIG. 133–b.

too much tamping at the ends of the rail in raising the joints which had become low. The track depression throughout the middle of the length of the rail under the static-load tests was 0.70 in. under the rear driver of the Atlantic locomotive, about double that found after the track had been properly tamped up, the latter being about the same as that which was found in ordinarily good track. Figs. 134 to 136 give the speed-stress diagrams for the Atlantic locomotive at this low spot. The number of observations is smaller than those averaged in the tests on the prepared stretches of track, but it is large enough to seem to be representative of the action of the track here. At 5 miles per hour the stress in rail is about 15% higher than for good track, and, at 60 miles per hour, from 20 to 35% higher. At the latter speed, values at the front driver were found as high as 43% more than for good track. In each case the values plotted are the averages of those obtained with the three instruments on the low rail for three or more runs.

Tests were made on the track which leads from the freight yards at *N*, in Fig. 26. This track has 6 in. of cinder ballast on a light loam soil. The ties were in good condition and the tie spacing uniform. Although not recently tamped, the track was in fair condition. Fig. 137 gives the speed-stress diagrams for the Mikado locomotive. The stresses are higher than those found with 6 in. of ballast on the main-line track, averaging, perhaps, 12% higher, and under some wheels being 18% or more higher. A rather marked characteristic of these diagrams is the unusual variation in values at the different wheels and the different speeds; individual results are found which are much higher than the average values plotted in the diagrams.

Although this report will not take up the observed action of the ties and ballast, and the relation of their properties to the properties of the track, it may be remarked here that the prepared main-line test track, which is ballasted with broken stone, seems to show little difference between the stresses in rail on track which has 6 in. of ballast and on track which has 12 in. The results are more erratic and more variable in the former case than in the latter, and, of course, the question of the time intervals between necessary track surfacing is not here considered. The track with 24-in. ballast was noticeably stiffer than either of the foregoing, giving smaller track depressions under load and a larger value of the modulus of elasticity of rail-support. The diagrams also show that the track on 24-in. ballast has somewhat smaller stress in the rail than that on lighter ballast.

To show that conditions of the locomotive may greatly affect the amount and distribution of the stresses in the rail, an instance which occurred in one of the early tests may be cited. A test was being made to compare the results obtained by the strain gauge and the stremmatograph. The stremmatograph record showed almost no stress under the third driver of the Mikado locomotive. Strain gauge read-

SPEED-STRESS DIAGRAMS FOR LOW SPOT IN TRACK.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD.
WITH ATLANTIC LOCOMOTIVE.

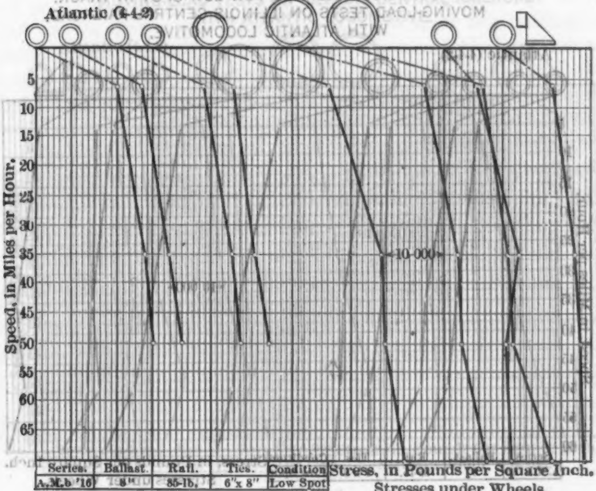


FIG. 134–a.

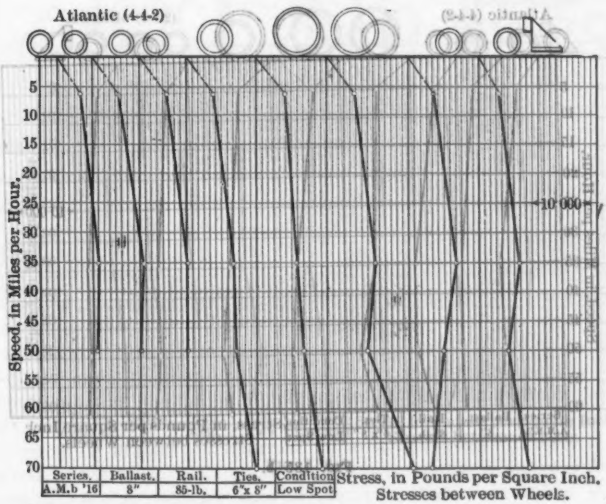


FIG. 134–b.

SPEED-STRESS DIAGRAMS FOR LOW SPOT IN TRACK.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD,
WITH ATLANTIC LOCOMOTIVE.

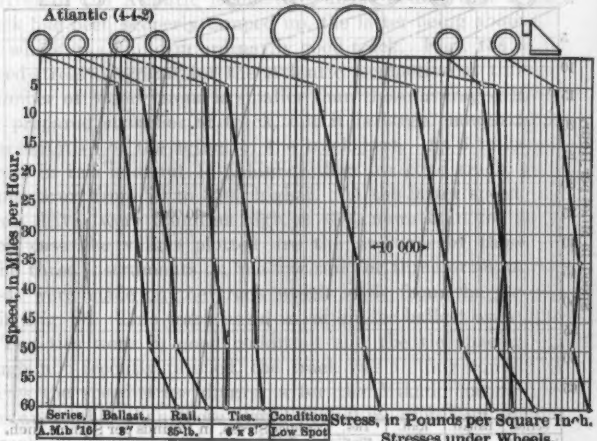


FIG. 135–a.

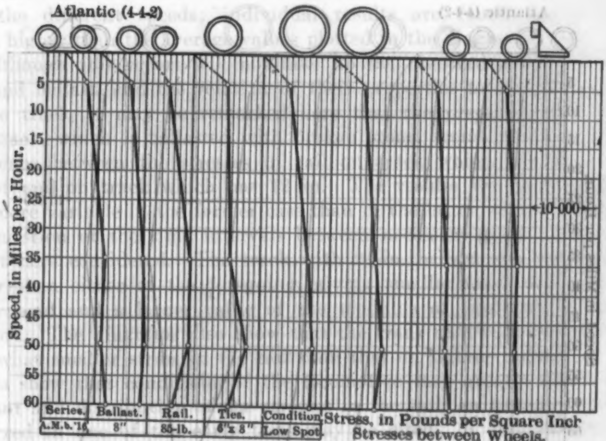


FIG. 135–b.

SPEED-STRESS DIAGRAMS FOR LOW SPOT IN TRACK.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD.
WITH ATLANTIC LOCOMOTIVE.

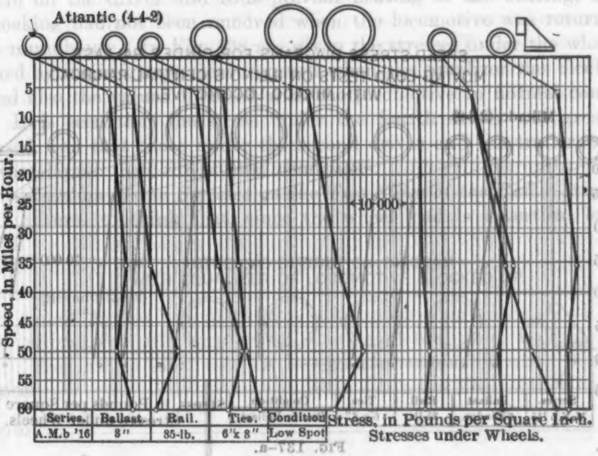


FIG. 136–a.

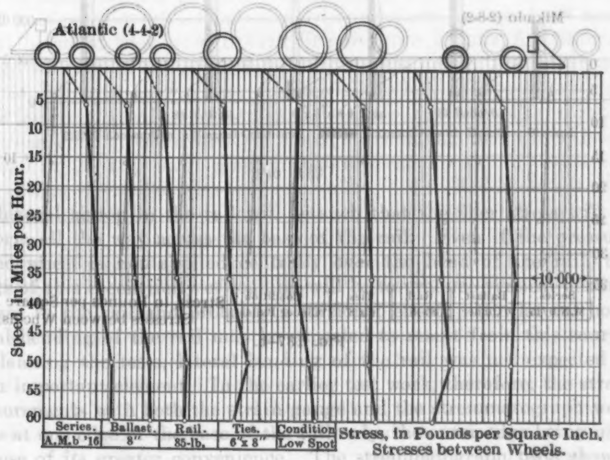
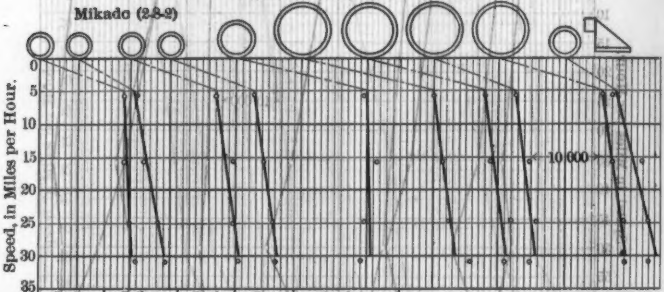


FIG. 136–b.

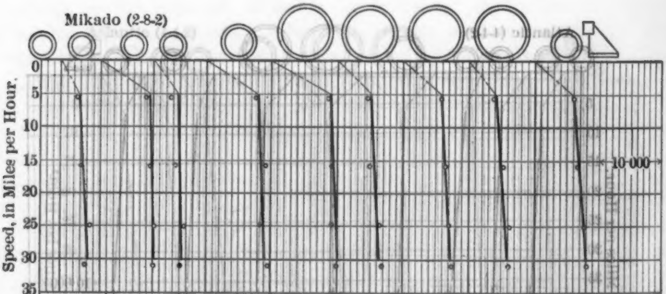
SPEED-STRESS DIAGRAMS FOR CINDER BALLAST.
MOVING-LOAD TESTS ON ILLINOIS CENTRAL RAILROAD
WITH MIKADO LOCOMOTIVE.



Series.	Ballast.	Rail.	Ties.	Condition.
M.N.b '15	6" Cinder.	85-lb.	6"x 8"	Cinder Ballast.

Stress, in Pounds per Square Inch.
Stresses under Wheels.

FIG. 137–a.



Series.	Ballast.	Rail.	Ties.	Condition.
M.N.b '15	6" Cinder	85-lb.	6"x 8"	Cinder Ballast.

Stress, in Pounds per Square Inch.
Stresses between Wheels.

FIG. 137–b.

ings made with static loading gave the same results. An examination showed that the equalizer for the third driver had been temporarily blocked up by the engineman while out on a run, in order to decrease the load on the driver and thus prevent heating of the bearing, and the blocking had not been removed when the locomotive was returned to the roundhouse. In Fig. 138 are given the stresses under the wheels obtained by strain gauge measurements when the equalizer was blocked up, and also the values obtained with the locomotive in normal condition. It is seen that the stress under the fourth driver was greatly increased, and that those under the second driver and the trailer were somewhat larger than for normal conditions.

53.—Relation of the Stresses on the Two Sides of the Rail.—It may appear natural to think that, since the wheel load is a vertical load,

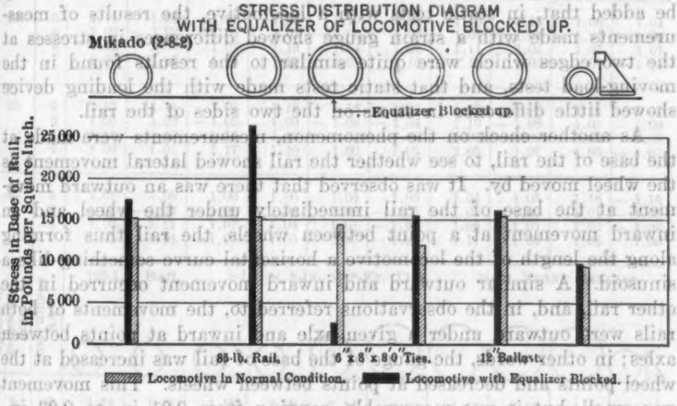


FIG. 138.

the flexural action in the rail will be such that the fiber stress will be uniform all the way across the base of the rail. Even if the pressure of the wheel is not applied centrally over the head of the rail, the effects of this eccentricity of loading will be to produce vertical flexural stresses in the rail and probably a torsional effect; little lateral horizontal bending of the rail may be expected to result from this source. In planning the tests, lateral bending of the rail was not expected to be an important element. In the earlier test work, therefore, the strain measurements with both the strain gauge and the stremmatograph were made at one edge of the base of the rail only, the outer edge being used because of its greater convenience. The stremmatograph tests showed marked irregularities and seemingly erratic results, which were unexplainable on the ordinary assumption, and steps were taken to find

the source of the variations. Tests were made with the position of the stremmatograph reversed on the rail, in order to measure the deformation at the inner edge, and the results were compared with those found with the measurements at the outer edge of the rail, all other conditions as to track, position of locomotive, and speed remaining the same. The average of the stremmatograph measurements at the two edges of the base of rail was found to be fairly uniform and consistent for runs which were comparable, but the stress at one edge was usually different from that at the other, sometimes more than twice as great. Generally speaking, the higher value was found at the outer edge of the rail. The instruments already made were then rebuilt in such way that each stremmatograph measured the deformation both at the inner and outer edges of the base of rail, and this improvement was followed in instruments made thereafter. It may be added that, in static tests with a locomotive, the results of measurements made with a strain gauge showed differences in stresses at the two edges which were quite similar to the results found in the moving-load tests, and that static tests made with the loading device showed little difference in stress on the two sides of the rail.

As another check on the phenomenon, measurements were made at the base of the rail, to see whether the rail showed lateral movement as the wheel moved by. It was observed that there was an outward movement at the base of the rail immediately under the wheel and an inward movement at a point between wheels, the rail thus forming along the length of the locomotive a horizontal curve something like a sinusoid. A similar outward and inward movement occurred in the other rail, and, in the observations referred to, the movements of both rails were outward under a given axle and inward at points between axles; in other words, the gauge of the base of rail was increased at the wheel points and decreased at points between wheels. This movement was small, but it was measurable, ranging from 0.01 in. to 0.03 in., with the 85-lb. rail in the preliminary test referred to. It will be seen, later, that the movement of the rail is not always of the character just described, there being several forms of movement.

With an outward bending of the rail at a point under a wheel load, the tensile stress at the outer edge of the base of rail is increased over that due to vertical bending, and with an inward bending at a point between wheels, the compressive stress at the outer edge of the base of rail is also increased, the stress at the inner edge of rail being decreased in both cases. In the head of the rail the effect on the stresses is reversed, but, as the head is narrower than the base, the difference in stresses at the two sides of the head of the rail is smaller than that found at the base.

Table 10 gives values of the ratio of the stress at one edge of the base of rail to the mean of the stresses at the two edges. Where the

TABLE 10.—VALUES OF THE RATIO OF THE STRESS AT ONE EDGE OF THE BASE OF RAIL, TO THE MEAN OF THE STRESSES AT THE TWO EDGES. PREPARED TEST SECTIONS OF TRACK.

85-LB. RAIL 6-IN. × 8-IN. × 8-FT. TIES 12-IN. STONE BALLAST

Speed, in miles per hour.	Instrument No.										
5	2	1.12	1.16	1.10	1.15	1.09	1.10	1.18	1.16	1.18	1.05
	3	1.13	1.07	1.05	1.11	1.05	1.00	1.02	— 1.04	1.03	1.06
	4	— 1.03	1.04	1.03	1.11	1.04	1.09	1.08	1.08	1.00	1.01
	7	— 1.18	1.10	1.09	1.15	1.02	1.07	1.10	1.02	1.09	— 1.07
15	2	1.11	1.06	1.05	1.13	1.10	1.08	1.15	1.12	1.08	1.04
	3	1.02	— 1.02	— 1.01	1.07	— 1.01	1.02	1.07	— 1.04	— 1.01	— 1.05
	4	1.12	1.09	1.07	1.13	1.04	1.09	1.11	1.02	1.07	1.01
	7	1.07	1.07	1.04	1.17	1.05	1.10	1.11	1.02	1.07	— 1.14
25	2	1.18	1.11	1.07	1.19	1.12	1.02	1.10	1.08	1.11	— 1.07
	3	1.05	1.03	— 1.05	1.00	— 1.08	— 1.04	1.10	— 1.08	1.01	— 1.09
	4	1.12	1.10	1.10	1.13	1.07	1.02	1.07	1.09	1.03	1.08
	7	1.09	1.01	1.17	1.10	1.00	1.05	1.05	1.07	1.05	— 1.07
35	2	1.01	1.05	1.00	1.00	1.04	1.06	1.04	1.05	1.18	— 1.15
	3	1.03	— 1.02	— 1.01	1.03	1.00	— 1.06	— 1.05	1.02	— 1.01	— 1.06
	4	1.10	— 1.11	1.07	1.04	1.01	— 1.02	1.05	— 1.01	1.00	— 1.03
	7	1.05	1.04	1.09	1.05	1.07	— 1.02	1.11	— 1.06	1.02	1.00

125-LB. RAIL 6-IN. × 8-IN. × 8-FT. TIES 24-IN. STONE BALLAST

Speed, in miles per hour.	Instrument No.										
5	8	1.15	1.11	1.18	1.09	1.14	1.12	1.11	1.05	1.20	1.14
	9	1.10	1.10	1.02	1.14	1.13	1.11	1.20	1.17	1.11	1.09
	11	1.07	1.09	1.05	1.15	1.14	1.14	1.26	1.23	1.13	1.13
	12	1.05	1.05	1.09	1.21	1.20	1.22	1.25	1.22	1.25	1.12
15	8	1.08	1.05	1.10	1.04	1.07	1.05	1.06	1.03	1.09	— 1.01
	9	1.11	1.09	1.01	1.13	1.10	1.12	1.26	1.15	1.15	1.13
	11	1.07	1.05	1.02	1.12	1.15	1.19	1.30	1.24	1.19	1.20
	12	1.17	1.11	1.24	1.22	1.24	1.16	1.37	1.32	1.31	1.15
25	8	1.07	1.02	1.02	— 1.02	1.02	1.07	— 1.02	1.06	— 1.03	— 1.01
	9	1.18	1.12	1.04	1.18	1.10	1.09	1.26	1.25	1.19	1.13
	11	1.12	1.12	1.06	1.16	1.10	1.11	1.23	1.19	1.24	1.13
	12	1.25	1.22	1.13	1.24	1.11	1.19	1.17	1.20	1.23	1.12
35	8	— 1.03	— 1.06	1.06	— 1.03	— 1.01	— 1.07	— 1.07	1.00	— 1.10	1.03
	9	1.09	1.06	1.03	1.09	1.07	1.04	1.25	1.15	1.09	1.04
	11	1.12	1.12	1.06	1.15	1.08	1.10	1.15	1.07	1.12	1.11
	12	1.10	1.08	1.14	1.20	1.10	1.22	1.11	1.22	1.17	1.28

minus sign is given, the stress at the inner edge is the greater; otherwise, that at the outer edge is the greater. The table is a sample of the data obtained (the complete data being very voluminous), and is probably representative of the observations. The values given are those found by individual instruments; averages of several instruments would mask the effect of variations in track and of variations in movement of locomotive. In looking over the table, it should be borne in mind that the runs were made in such a way that the counterweight of the front driver, on the side of the track on which there were three instruments, was at its lowest point as it passed the middle instrument, and that the succeeding wheels made their records at a later time, and, therefore, the measurement of stresses for the several wheels was not made simultaneously. Naturally, the values cover quite a range. There are many ratios as high as 1.30, and a number as great as 1.50 (which means that the stress at one edge of the base is 1.86 and three times as great, respectively, as the stress at the other edge). The few ratios higher than 1.75 doubtless may not be fully reliable.

The ratios at points midway between wheels (not given in the table) run to even higher values, and are more discordant, as might be expected from the fact that the magnitude of the stress is less than that at points under the wheels. The difference between the numerical values of the stresses at the two edges, as may be expected, is in general less at the point between wheels than at points under the wheels.

The phenomenon of difference in stress at the two sides of the rail is found at all speeds. The magnitude of the stress at either edge of the base of rail varies with the speed, generally increasing with increase of speed, though sometimes the stress at one edge decreases with the speed. In general, the ratio of the stresses at the two edges of the base of rail is not noticeably different at high than at low speeds. In fact, the changes in stress with speed are quite consistent at each edge, as is shown in Fig. 139.

It appears that one of the elements which affect the differences in stresses developed at the two edges of the base of rail is the condition of the track. Higher ratios are found more frequently in track not recently tamped. An excellent example of the effect of the condition of track is seen in the observations at the two locations having a tie in the partly decayed condition previously referred to. The position of the instruments with reference to the poor tie is shown on Fig. 131. The ratio of the stress at edge of rail to the mean of the stresses at the two edges is given in Table 11. Each value given for a single instrument is the average of the results of several runs at a given speed; individual runs give higher ratios. In the test at *I*, Fig. 26, for Instrument No. 2, which was next to the decayed tie, the average of the ratios under the drivers and trailer for all speeds is 1.35 (about 80 readings in all). The average of the highest values found at each

speed for all the drivers and trailer is 1.46 (about 16 readings included in getting this average). Stated in the other way, this means that the stress at the outer edge of the base of rail is about 2.1 times the stress at the inner edge when the ratio is 1.35, and 2.7 times when the ratio is 1.46. For Instrument No. 3 (at the tie space ahead of No. 2), the average ratio at all speeds is 1.26, the highest being 1.36. For Instrument No. 7, which was on the other rail and opposite Instrument No. 3 (this end of the decayed tie being sound), the variation in stresses is the same as at No. 3 (that is, the rail bends outward); the average of the ratios at all speeds is 1.10, with a maximum of 1.14. For Instrument No. 4, ahead of No. 3 and on the other side of the decayed tie, the ratio is 1.16 and runs sometimes positive and sometimes negative. For this instrument, the ratios given by the records at the time the

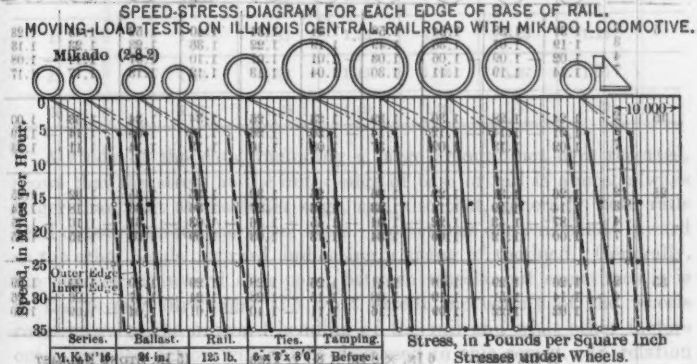


FIG. 139.

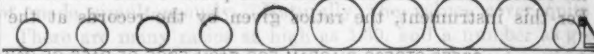
instrument was midway between wheels are high, the average for all the wheels and all the runs being 1.48, with maximum values of 1.67.

In the case of the other test location at a partly decayed tie (*L*, Fig. 26), the instruments were placed on one rail only. The values in Table 11 show highest values for Instrument No. 7, although No. 4 gives nearly as high an average. One driver gives an average ratio at all speeds of 1.43, with a maximum of 1.50 at one speed.

High ratios of the stress at edge of base of rail to the mean of the stresses at the two edges were found in the case of track ballasted with cinders. This track (*N*, Fig. 26) was not in as good surface and alignment as that of the main line. Table 12 gives the ratios under the drivers and trailer at the several speeds. It is seen that ratios as high as 1.35 were not infrequent. In Fig. 140 the stresses at the two edges of the rail, found by averaging the results from the four instruments, are shown for the four speeds used. It is seen that for each wheel the

stress at one edge of the base of rail is in general consistently larger than at the other. The averages of all the instruments at 30 miles per hour give stresses of 31 000 lb., and the records of individual instruments were as high as 35 000 lb. (the latter being, of course, the average of several runs).

TABLE 11.—VALUES OF THE RATIO OF THE STRESS AT ONE EDGE OF THE BASE OF RAIL TO THE MEAN OF THE STRESSES AT THE TWO EDGES. TIE IN PARTLY DECAYED CONDITION.



Speed, in miles per hour.	Instrument No.										
		85-LB. RAIL		6 IN. × 8 IN. × 8-FT. TIES Location I				6-IN. STONE BALLAST			
5	2	1.31	1.38	1.34	1.58	1.24	1.34	1.40	1.52	1.45	1.23
	3	1.19	1.07	1.36	1.49	1.19	1.22	1.36	1.23	1.23	1.13
	4	1.02	— 1.09	— 1.06	1.08	— 1.01	1.07	1.10	1.07	1.14	— 1.08
	7	1.14	1.19	1.11	1.30	1.04	1.13	1.12	1.13	1.17	1.17
15	2	1.24	1.28	1.30	1.39	1.23	1.26	1.34	1.34	1.53	1.00
	4	1.01	— 1.14	— 1.12	1.14	— 1.20	1.26	1.18	1.00	1.14	— 1.19
	7	1.02	1.13	1.09	1.37	1.08	1.10	1.13	1.14	1.11	1.33
25	2	1.26	1.31	1.29	1.36	1.26	1.30	1.35	1.35	1.32	1.25
	3	1.14	1.19	1.14	1.33	1.16	1.27	1.36	1.36	1.18	1.14
	4	— 1.37	— 1.43	— 1.22	— 1.04	— 1.45	— 1.16	— 1.08	— 1.61	— 1.10	— 1.45
	7	1.00	1.12	1.06	1.34	1.13	1.08	1.09	1.05	1.15	— 1.35
35	2	1.26	1.29	1.18	1.41	1.26	1.28	1.29	1.39	1.34	1.39
	4	— 1.14	— 1.51	— 1.19	— 1.08	— 1.15	— 1.28	— 1.24	— 1.47	1.06	— 1.26
	7	1.02	1.22	1.09	1.16	1.11	— 1.10	— 1.01	1.04	— 1.09	1.20
		85-LB. RAIL		6 IN. × 8 IN. × 8-FT. TIES Location L				15-IN. STONE BALLAST			
5	7	— 1.03	1.05	— 1.01	1.25	1.11	— 1.14	1.07	1.00	1.29	1.11
	4	1.28	1.33	1.34	1.37	1.26	1.23	1.35	1.32	1.18	1.17
	2	1.01	1.37	1.11	1.29	1.13	1.16	1.22	1.28	1.27	1.05
15	7	1.06	1.07	1.13	1.38	1.11	1.01	1.11	1.08	1.30	1.11
	4	1.15	1.09	1.09	1.31	1.20	1.22	1.37	1.25	1.31	1.19
	2	1.20	1.11	1.21	1.39	1.21	1.25	1.41	1.23	1.07	1.01
25	7	1.12	1.11	1.11	1.40	1.07	1.07	1.24	1.10	1.27	1.14
	4	1.19	1.09	1.06	1.42	1.11	1.25	— 1.40	1.25	1.41	1.23
	2	1.27	1.24	1.28	1.56	1.12	1.33	1.47	1.37	1.37	1.24
35	7	1.14	— 1.06	1.03	1.22	1.06	1.06	1.19	1.11	1.14	1.03
	2	1.32	1.28	1.44	1.48	1.15	1.27	1.50	1.28	1.31	1.30

Some high values of ratios found on the prepared test track when in good condition are given in Table 13 which gives the results of a single run with a Mikado locomotive at a speed of 35 miles per hour.

It is seen that at Instrument No. 4 all wheels except one give high ratios, the highest being 1.75 (the stress at the outer edge in this case being 35 000 and that at the inner edge 5 000 lb. per sq. in.). At Instrument No. 2, the stresses at the inner edge are usually higher than those at the outer.

TABLE 12.—VALUES OF THE RATIO OF THE STRESS AT ONE EDGE OF THE BASE OF RAIL TO THE MEAN OF THE STRESSES AT THE TWO EDGES. TRACK WITH CINDER BALLAST.

Speed, in Miles per Hour.	85-lb Rail.	6"x 8"x 8'0" Ties.		8" Cinder Ballast.		
	Mikado 					
5	1.24	1.22	1.31	1.26	1.14	1.09
15	1.22	1.17	1.35	1.30	1.25	1.22
25	1.13	1.24	1.45	1.32	1.27	1.05
30	1.17	1.28	1.42	1.33	1.31	1.20

A study of the data, including a large amount not here reported, shows that the stress at the outer edge of base of rail is almost always larger than that at the inner edge, and also that, generally, both rails bend outward under the wheel loads. Sometimes one rail bends inward as the other bends outward, indicating a swing of the load. Generally, the bending is inward at a point between wheels, but sometimes it is in the reverse direction. To what extent the coning of the wheels contributes to the outward bending of the rail and what other factors enter into the action will not be discussed in this report. In relation to this matter, it should be noted that, for the tests with the loading jacks, very little difference was found in the stresses at the two sides of the rail.

It seems evident, from a study of all the data, that stresses at the edge of base of rail 30% greater than those due to vertical load may be expected in what would be considered good track, and that occasional values considerably higher than 30% are probable. With track in poor condition, the lateral bending of the rail is even greater. This means that the lateral strength of the rail is an important element in track resistance. For the various rail sections used, the section modulus, $\frac{I}{c}$, with reference to a vertical axis, averages about $\frac{23}{100}$ of that for a horizontal axis. The lateral bending moment, then, is much less than the vertical, but its effect in increasing the fiber stress is such that it needs consideration in the determination of stresses in rail.

54.—Variation in Measured Stresses, and the Relation between Average and Maximum Stress.—The stress developed in the rail under moving loads applied through locomotive and car wheels may not be expected to have the same magnitude for different applications of load at the same place, or for different places having nominally the same conditions. What variations from the average stresses already reported may be expected to occur and what maximum stresses may be expected are matters of importance. To arrive at a judgment on these questions will require a study of the variation in the observed values and of the probable errors of observation. Having determined the limiting error of observation which is probable and also the variation in the observed values which may be expected, a value of the probable limiting variation in the actual stress developed in the rail may be approximated, which will assist in judging what maximum stress may be expected to occur.

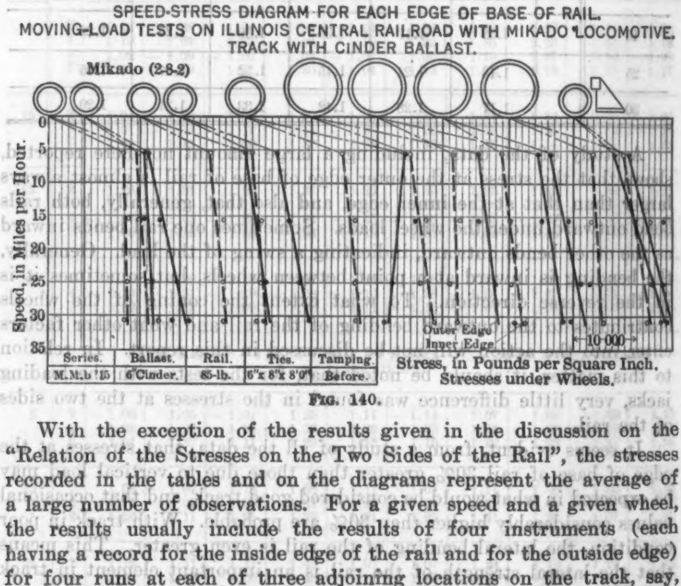


Fig. 140.

With the exception of the results given in the discussion on the "Relation of the Stresses on the Two Sides of the Rail", the stresses recorded in the tables and on the diagrams represent the average of a large number of observations. For a given speed and a given wheel, the results usually include the results of four instruments (each having a record for the inside edge of the rail and for the outside edge) for four runs at each of three adjoining locations on the track, say, counting out imperfect records, 90 observations in all; the average of the 90 observations is recorded as the value for the given wheel at the given speed. In the case of the tests on track at unsound ties and at low spots, the values given are the averages for one instrument, say 8 observations in all for each wheel at a given speed. For the comparison of stresses on the two sides of the rail, the values given are

the average of the records on a stremmatograph disk at one side of the rail at three locations, say, 12 observations at each speed and 48 observations for all speeds. It is evident that the individual observations will vary from the average result reported. Part of this variation will be due to errors in observation, and part will represent actual variation in the stresses in the rail.

TABLE 13.—VALUES OF THE RATIO OF THE STRESS AT ONE EDGE OF THE BASE OF RAIL TO THE MEAN OF THE STRESSES AT THE TWO EDGES FOR A SINGLE RUN ON A PREPARED TEST SECTION OF TRACK.

Speed, in Miles per Hour.	Instrument No.	85-lb. Rail.			6"x 8"x 8'0" Ties.				6" Stone Ballast.		
35	2	-1.06	-1.15	-1.23	-1.20	-1.25	-1.01	-1.16	-1.03	-1.13	-1.14
	3	-1.20	-1.13	1.07	1.00	1.17	1.15	-1.05	1.23	1.12	-1.05
	4	1.20	1.06	1.49	1.23	1.48	1.73	.1.11	1.00	1.31	1.00

Variations in the actual stress in the rail under nominally the same conditions may be due to differences in the way the locomotive wheels and car wheels are applied to the track in runs made at different times, as, for illustration, that produced by nosing or rolling, or by variations in the action of equalizers and springs; and they may be due to differences in the way the rail-support (ties, ballast, etc.) acts under changes in method of application of load, or to actual difference in the resisting action of the track support at two locations having seemingly the same conditions.

The sources of observational errors are: (1) inherent characteristics in the design of the instruments, such as that in the stremmatograph used in part of the 1915 tests; (2) looseness or play in bearings of record disk, or somewhere in the clamps; (3) accidental conditions of operation, such as result from variable jolts or blows on the track, and from variable inertia effects in the instrument; and (4) errors in reading the record.

The first-mentioned source may give errors which are proportional to the stress, or nearly so. This sort of instrumental error was found in the stremmatograph used in part of the 1915 tests. It is believed that the design of the later instruments avoided errors of this kind which were of any magnitude. The second source of instrumental error gives variable accidental conditions in the use of the instrument. In the stremmatograph used, the presence of this condition, when sufficient in amount, could be detected by an examination of the record in the field, and the instrument could be adjusted at once. Usually, a variation in the action of the instrument, equivalent to a stress of 600 lb. per sq. in., could be detected. When it was seen that the variation

amounted to more than 1 000 lb. per sq. in., the observation in question was discarded. For variations between these two general limits, the observation was used, and an allowance was made, and the error from this source may not be expected to reach more than 600 lb. per sq. in. for an individual observation. The variations from the third source, accidental conditions, frequently could be told from the characteristics of the record. When such a variation was marked, the observation was discarded. Notwithstanding this, there must exist in the results obtained from the records many variations which are due to accidental instrumental errors. Of the fourth source, that of reading the record,

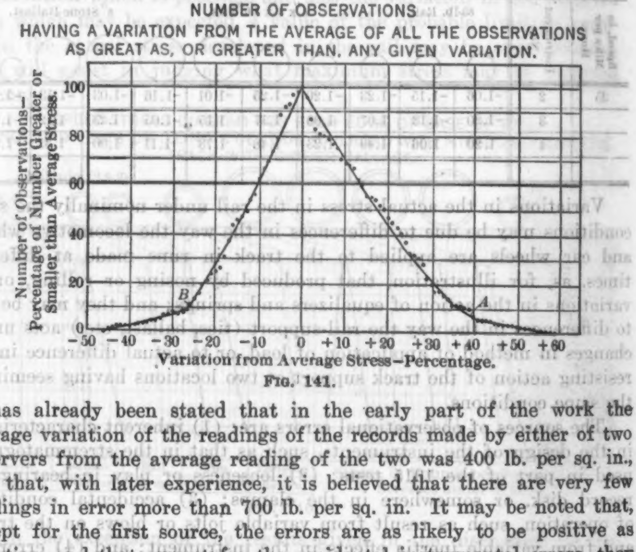


FIG. 141.

it has already been stated that in the early part of the work the average variation of the readings of the records made by either of two observers from the average reading of the two was 400 lb. per sq. in., and that, with later experience, it is believed that there are very few readings in error more than 700 lb. per sq. in. It may be noted that, except for the first source, the errors are as likely to be positive as negative, and that, except for the accidental instrumental errors, they are largely independent of the magnitude of the stress; that is, they are not greater for large than for small stresses.

For the study of the variations, a number of sets of test results were chosen at random. These included tests made with 85-lb., 100-lb., and 125-lb. rail at the several speeds used, for each edge of base of rail, and for the mean of the two edges. Diagrams were plotted for each set, in order to show the relation between the magnitude of the variation from the mean and the number of observations. Fig. 141 shows one of these. The horizontal scale gives the variation from the average value of all the observations as a percentage of the average stress. The vertical scale gives the number of observations having a variation from the average value as great as, or greater than, the variation in

question. At two points on these diagrams, such as A and B, a rather marked change in slope or break in the curve was found. Although the several diagrams differed somewhat in the variation indicated at these characteristic points, the location of the points did not vary much from a little less than 10% of the number of observations, and the average magnitude of the variation at these points was $\pm$ 38% of the average stress. Outside of these points, the number of observations corresponding to the variations became smaller, but there were always several values at or beyond $\pm$ 50% grouped in such a way as to seem well substantiated. Beyond these the observations were few and very irregularly placed. It may be stated, then, that, in the sets studied, 10% of the observations gave variations from the average stress of as much as $\pm$ 38% of the average stress, and that always there were seemingly well substantiated variations as great as $\pm$ 50 per cent.

In the several sets of results studied, the mean variation of the individual observations from the average stress did not differ much; it averaged $\pm$ 22 per cent. Similarly, the probable error of a single observation, calculated by the theory of errors, was nearly the same in all the sets, averaging $\pm$ 18 per cent. The diagrams, such as that in Fig. 141, are very similar in form to the curve of the probability integral, as they may be expected to be, both because the errors of observation will follow the laws of chance and because the causes which produce variable stresses in the rail may also be expected to follow the laws of chance in the frequency of their application. The different sets of results which were studied had common characteristics; the variations at the line of 10% of number of observations ($\pm$ 38%), the values of the mean variation from average stress, and the probable error differed little in the several sets. They were about the same at all speeds, for all wheels, and for both edges of the rail. This is all characteristic of accidental differences.

A consideration of the source and probable amount of the observational errors leads to the view that in 90% of the observations the error of a single observation will be less than 15% of the observed stress, and, of course, generally much less than this. This estimate takes into account both the observational errors that are independent of the magnitude of the stress and those that may be proportional to it; this limit of error may be somewhat too small for low stresses, such as 5 000 to 10 000 lb. per sq. in., and somewhat too large for high stresses

If then it be assumed that the error of observation for the observed stress at which there are 10% of the observations giving greater variation (B, in Fig. 141) is $\pm$ 15%, and the variation of this observed stress from the average is taken as $\pm$ 38%, it will be seen that the actual value of the stress in the rail which was observed as 1 + 38% may be as small as $\frac{1.38}{1.15}$ $\times$ average stress, or 20% more than the

average; and, of course, it might be as large as $\frac{1.38}{0.85} \times$ average stress, or 62% more than the average. Similarly, for an observed stress 50% greater than the average, assuming that the limit of error of observation is $\pm$ 20%, the actual stress in the rail may be as small as 25% more than the average. It may be asserted, then, that, for nominally the same conditions of track and of load values of the stress in the rail, 20% greater than the averages reported in the diagrams and tables, or even 25% greater, may be expected not infrequently, and, of course, by the theory of chance, the probabilities are that these values may be exceeded considerably. In any effort to determine the maximum stress which may be developed in a rail, this excess of the individual values of the stress over the value obtained by averaging results must be taken into account.

55.—Method of Estimating Maximum Flexural Stress in Rail for Given Conditions of Track and Loading.—What maximum flexural stress may be developed in a rail under given conditions of track and loading is a matter of considerable interest. In making an estimate of the maximum stress, it will be well to start with the average stress which may be expected at low speeds for the best conditions of track, and introduce a series of factors covering the several sources of increase or variation from average stress at low speeds. Such an equation for flexural stress in the rail is

$$f = f_0 (1 + a) (1 + b) (1 + c) (1 + d)$$

$$= \begin{pmatrix}\text{average stress}\\ \text{at 5 miles per}\\ \text{hour.}\end{pmatrix} \begin{pmatrix}\text{speed}\\ \text{factor.}\end{pmatrix} \begin{pmatrix}\text{lateral}\\ \text{bending}\\ \text{factor.}\end{pmatrix} \begin{pmatrix}\text{condition}\\ \text{of track}\\ \text{factor.}\end{pmatrix} \begin{pmatrix}\text{varia-}\\ \text{bility}\\ \text{factor.}\end{pmatrix}$$

These factors have already been discussed to some extent. The average stress at 5 miles per hour would depend on the locomotive loading and the nature of the track (rail, ties, ballast, etc.). For the "condition of track factor" the presence of low spots, defective ties, and other imperfect conditions must be considered, and, even on fairly well-kept track, this factor will need consideration. The Committee plans to discuss in another report the magnitude of maximum stress for various conditions of track and loading. It is evident that the maximum stress which may be developed in the rail under usual conditions of track and traffic may be much higher than the average stresses found for best track at low speeds, and that speed factor, lateral bending factor, condition of track factor, and variability factor should be recognized in making an estimate of the maximum stress to which the rail will be subjected.

56.—The Test Work on Ties, Ballast, and Roadway.—Considerable progress has been made in the experimental study of the action of ties, ballast, and roadway. Tests on track have included the measurements of the depression produced under load at various points below the tie and across and along the track and roadway. The measurement

of pressure in ballast with the pressure-capsule has given interesting data. Information has also been obtained with the recording pressure-capsules. Measurement of the flexure of the tie has been made. Before completing tests on this part of the problem, it was deemed wise to await the results of laboratory tests, in order to determine the principles governing the distribution of pressure through ballast material, as this information would guide the extension of the tests and be useful in interpreting the data obtained.

The laboratory tests on transmission of pressure through ballast material have advantages over tests made in the field, in that the conditions may be controlled more definitely and exactly and the tests repeated under identical conditions. In the experiments, several ties are spaced as usual over a given depth of ballast, and loads are applied on the rails. By having a number of pressure-capsules inserted in the ballast at the different depths and positions at which the measurement of pressure is desired and then changing tie spacing and other variables, information bearing on the problem is obtained. Tests have been made with broken stone, gravel, and sand as ballast material. Advantage has been taken of information and experience gained in tests on the transmission of pressure through granular material previously carried on in the laboratory. It seems evident, from the experimental data obtained, that quite definite laws govern the transmission and distribution of pressure through ballast materials, and that these principles have a very direct bearing on the effectiveness of different depths of ballast and different conditions of ties and tie spacing. The results of the investigation on ties, ballast, and roadway will be presented in another report.

Respectfully submitted,

The Committee to Report on Stresses in Railroad Track

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FINAL REPORT OF THE SPECIAL COMMITTEE ON MATERIALS FOR ROAD CONSTRUCTION AND ON STANDARDS FOR THEIR TEST AND USE*

WITH DISCUSSION BY MESSRS. E. DOW GILMAN AND J. O. PRESTON.

TO THE AMERICAN SOCIETY OF CIVIL ENGINEERS,

GENTLEMEN:

Your Special Committee on "Materials for Road Construction and on Standards for Their Test and Use" herewith respectfully submits a Final Report covering its work from the date of its appointment by the Board of Direction.

Through the courtesy of the Board, successful meetings have been held in the Society House, for one or two days subsequent to each Annual Meeting of the Society, for the discussion of matters of interest connected with the work of the Committee. These meetings have been well attended, usually by more than 200 at a session, by highway authorities outside of the Society as well as by members of the Society itself. It is felt that valuable, as well as authoritative, information and criticism has thus been secured to the Profession as well as to the Committee.

Your Committee has rendered progress reports annually since 1911, and in the accompanying report has given careful consideration to its earlier conclusions and the criticism and discussions submitted in connection therewith.

The Committee has been made aware of a widening of the interest in highway work among members of the Society and others not experienced or even thoroughly trained in highway engineering. In the accompanying report it has been deemed wise, for the benefit of this new and wider interest, as well as for the purpose of making this Final Report comprehensive, to include conclusions or statements that may seem primary or trite to highway experts. Further, in order to render many of the earlier and present conclusions of the Committee

* Presented to the Annual Meeting, January 16th, 1918.

more intelligible to the wider interest mentioned, as well as to demonstrate their practical application in many cases, and to suggest the variables still remaining to be determined in specific instances, it has been thought wise to submit in this report principles underlying the drafts of specifications.

Your Committee believes that competent highway engineers may meet successfully the demands of any particular case by following these principles and eliminating the variables, necessarily left therein in order to express conclusions of general application, after proper consideration of the local factors affecting them.

However, it does not believe that these conclusions as to specifications will offset a serious lack of knowledge or experience in highway engineering or furnish a discriminating ability otherwise lacking. It also wishes to warn against any attempt to reduce the principles for specifications to the establishment of "the one best pavement" for any conditions.

One of the main activities of the Committee since its foundation has been toward securing accuracy and uniformity in the recording of data connected with highway work. To this end the Committee has suggested tests and analyses of materials in detail, records of traffic, costs, construction and maintenance details, and definitions of terms used with peculiar meanings in highway work. When it began its work in 1909, the status of affairs in these respects was found to be deplorable. Since then the work of this Committee and of committees on similar lines from other societies, as well as the work of some individuals, has materially improved the situation. Your Committee has made some suggestions which have been quite widely adopted. On the other hand, it has not hesitated to endorse the conclusions of others where such action seemed proper, hoping thereby to aid materially in the accomplishment of that desirable uniformity mentioned above. It hopes that its conclusions may be generally approved, even at the sacrifice of some minor differences of opinion, in order that uniformity in these matters, with resulting benefit, may be secured at the earliest possible date. The Forms, Analyses, and Test Details will be found in the Appendices hereto attached.

In its previous reports, your Committee has suggested a number of matters worthy of investigation. Sufficient time has not yet been available to the Committee for reaching satisfactory conclusions on all these points, and it has seemed to be a grave question whether or not it would be able, with the facilities at its disposal, to reach such conclusions within a reasonable period. Your Committee, therefore, invites the attention of all those interested to the following list of investigations it thinks worthy of prosecution, and expresses the hope that some authoritative data, which will throw light on these points, may in some way be secured at an early date:

The effect of various kinds of earth or soil on bituminous carpets under known conditions of traffic;

The efficiency of the use of asphaltic oils for surface treatments on gravel and broken-stone roadways;

The standardization of, and determination and expression of the consistency of, bituminous materials which preferably may permit of its use on all the wide range of such materials, from liquid to solid, and of the inter-relation existing in certain, if not all, cases;

The maximum and minimum quantities of free carbon that can be successfully allowed in tars under known conditions of other constituents of the tar, of climate, of traffic, and of methods of use;

Methods of determining the adhesive strength of bituminous cements;

The advisability of incorporating in bituminous pavements, built by the penetration method, bituminous material in excess of the minimum quantity necessary for the production of satisfactory results;

The determination of the amount of internal wear of the materials comprising a macadam roadway under known conditions of traffic, and the effects on the internal wear of increasing or diminishing the size of the stone in the courses;

The determination of the causes of cracks in concrete and brick roadways, and the best methods to be used to reduce such cracks to a minimum;

Rolling or other methods calculated to increase the surface density of cement-concrete pavements;

The relative merits of one-course and two-course plain cement-concrete pavements, and plain and reinforced cement-concrete pavements;

The relative efficiency of bituminous and cement fillers for block pavements;

The determination of the quality—which might be called its hardness or resiliency—of wood used for paving purposes;

The continuance of traffic census, and study of the results obtained, to determine the effect of motor traffic on various roadway surfaces, and especially with reference to the speed of the vehicle, and the establishment and expression of any relation between the traffic and the life or cost of any pavement;

The further study of suitable joints for block pavements on steep grades;

Proper methods of sampling highway materials.

Many points are as yet undetermined, and seem improbable of early settlement. Conclusions, in many instances, will be hastened by a prompt agreement among investigators that uniformity is fully as necessary as accuracy in their work, and that the use of methods of analysis conforming to those carefully specified in this report will produce better and quicker results than a wide use of various methods, more or less accurate in some details, but not consonant with those of the Committee. The conclusions of your Committee have been reached and expressed after careful consideration of the work of committees of other societies and with intimate knowledge of the details of that work through the membership on these other committees of members of this Committee.

Your Committee is impressed by the importance of the factor of costs, both as to construction and maintenance of highways, and of the need for comparable records of such data. The present situation in this respect is unfortunate, there being found available but few records of costs uniformly or logically compiled. Hence comparisons are difficult, if not impossible. Your Committee, therefore, invites special attention to its conclusions and forms in the matters of costs. The same may be said concerning traffic records. The propriety of a material or a form of highway construction is often determined by its cost under known traffic conditions. To reduce both cost and traffic data in different cases for purposes of comparison requires that they be expressed in some uniform, definite, and intelligible manner.

Your Committee submits the specific conclusions it has reached, as follows:

General Conclusions.

Selection of Roadway Surface.—The Committee believes that, with the development of highway work, it should be constantly more apparent that one of the greatest problems to be solved by highway engineers is the proper selection of the particular material and form of construction to be used which will most efficiently meet the conditions of any particular case, and that progress will be hastened by complete recognition of this fact. It recommends that the selection of the kind of crust or pavement be based on the following factors, the special value of which may be estimated in each case under the local conditions of traffic, surroundings, climatic conditions, and physical and financial resources, both as to construction and maintenance, with proper regard for probable or possible changes in these circumstances: First cost, maintenance cost, annual cost, ease of maintenance, durability, cleanliness, tractive resistance, slipperiness, favorableness to travel, sanitariness, noiselessness, and appearance.

Traffic.—Your Committee desires to emphasize the fact that experience has demonstrated the value of a traffic census taken both

preliminary and subsequent to the construction of a highway. The traffic census should be considered one of the most important variable factors in the solution of the problem of the selection of that type of construction best suited to local conditions, considered from the standpoints of both economy and efficiency. In connection with the census returns on any highway, should be considered the traffic on cross and parallel highways and the effect of improvement of these highways on the traffic of the highway under construction. The bald return of a traffic census, however, should not be the sole basis of the selection of the type of construction, but should be considered a guide in determining the value of the type to be adopted. In considering the effect of traffic and its relation to the design and cost of maintenance, it is necessary to take into account the speed as well as the weight of the vehicles.

Costs.—The Committee recommends that highway departments, in making their reports, adopt a tabular form similar to the one submitted in Appendix A.

Grades.—A choice of the material, or methods of using a particular material, may be affected by the grades as fixed. Certain materials or results of using materials for highway surfacings will be unsatisfactory outside of certain limits of grades. Conservative practice fixes the maximum limits for satisfactory results with grades as follows:

Kind of Roadway.		Maximum Grade.
Asphalt block		8.0%
Bituminous surfaces		6.0%
Bituminous concrete		8.0%
Bituminous macadam		8.0%
Brick	cement filler	6.0%
	bituminous filler	12.0%
	"Hillside" block	15.0%
Broken stone		12.0%
Cement-concrete		8.0%
Gravel		12.0%
Sheet-asphalt		5.0%
Stone block	cement filler	9.0%
	bituminous filler	15.0%
Wood block		4.0%

The minimum grades allowable will depend on local conditions as to climate, type of construction, character and amount of traffic, conditions of underlying and adjacent soil, and such other circumstances as affect drainage. Except for roadways on fills, where the outside edges of the surfaces of the shoulders are at least 2 ft. above the level of the adjacent ground or water level, or except in cases

where the roadway is laid over sand of such a character that it never becomes water-logged, a longitudinal grade for the roadway of less than one-half of 1% should not be used for roads.

On streets where the smoothness and evenness of the roadway surface may be confidently expected to be the greatest, and where conformity to the proposed elevations of surface is more carefully sought and more accurately possible, a minimum grade of as low as one-quarter of 1% has sometimes given satisfactory results in an emergency; but a minimum of one-half of 1% would be better as an established standard.

Widths.—The width of the roadway to be built will be determined largely by local circumstances, but, in view of the recent, constant, and rapid increase of traffic on highways, both in number of vehicles and in size of loads, it will be in the interest of economy for designs of highways to be made with proper consideration of further increase.

Where motor traffic forms a considerable proportion of the total traffic likely to use a highway, the unit width of traffic lines to be considered is 9 or 10 ft., instead of 7 or 8 ft., as heretofore, because of greater clearance required for the safe passing of the units of such traffic.

Where bituminous pavements are laid, the edges need protection, and a sudden transition from the pavement to any soft shoulder material should be avoided by means of extra width, or of cement-concrete or other edges, or such reinforcement of the shoulder material as may be necessary.

The width of roadways of rigid material, such as cement-concrete or vitrified block, should be at least equal to what would be prescribed under local conditions for a less rigid surfacing. The great difference between the firmness of a rigid roadway surfacing and of material frequently available for the shoulders thereto, often makes it necessary for safety and convenience of traffic, as well as for economy of maintenance, that the rigid surfacing shall be built wider than would answer for a more flexible surfacing, such as water-bound macadam, for instance, under the same local conditions. For single-track roadways, the width of the pavement should not be less than 10 ft., and for two lines of traffic, it should not be less than 18 ft., unless exceptionally durable shoulders are provided. In a street or alley, the width will ordinarily be determined by the necessary location of the curb.

Too narrow a width of roadway encourages, if it does not compel, concentration of traffic to such an extent as frequently to make unfair demands on what would otherwise be a suitable and efficient material for the surfacing. This may be especially noticeable at abrupt changes in the lines of the highway, where any tendency toward the improper concentration of traffic into too narrow areas should be avoided, as

far as possible, by such adjustment or separation of lines, and adjustment of width, of crown, or of slope, of the roadway surfacing as will keep the strains of the surfacing material within reasonable limits.

Too great a width for the roadway surfacing is as unwise in many ways as too little. Excessive width not only results in unnecessary first cost and interest charges, but also in needlessly increased maintenance and cleaning costs. Further, especially in the cases of those pavements where at least a minimum amount of travel is needed to preserve the surface in good condition, an excess of width may result in the development of areas from which disintegration of the whole pavement may rapidly spread.

Thickness.—In determining the thickness of any road crust or pavement, there must be consideration of the character of the foundation and of the weight of the vehicles to be supported. Although the general practice has been, too often, perhaps, to use mass, for the sake of safety in the preparation of the pavement, it now appears evident that some waste has been incurred in this direction, and that a more scientific determination of the thickness is possible without sacrifice of safety and yet with economy. However, in view of the recent, constant, and rapid increase of the weight, and consequently of the strains, caused by the traffic, it will be in the interests of economy for all designs of highways to be made with proper consideration of further increases.

In considering the character and capabilities of the foundation relative to the forces coming thereon through a pavement, the condition of the foundation under the most adverse conditions likely to exist for withstanding the forces, and the traffic stresses present and probable, and the character of the surfacing itself, should be taken into account. An absorbent sub-grade material likely to become soaked with water so as to weaken its supporting power may require a thicker slab, or even the addition of an artificial foundation, in order to disperse properly the stresses from the surface of the pavement. For instance, if laid on a strong sub-grade and one likely to remain always in good condition, the minimum thickness of 6 in. for a broken stone roadway will be sufficient, and possibly even 4 in. will be enough where the vehicles passing over the road are comparatively light, that is, of 1 or 2 tons on four wheels. With a concrete slab, ordinarily, from 4 to 8 in. in thickness will suffice, though, in order to prevent the possibility of a sudden rupture of the slab on some sub-grades by an exceptional load, a uniform thickness above the minimum may be wise. Again, mass—that is, unusual thickness—may in some cases be desirable with the use of a minimum of cement, not only for reasons of economy, but also for the purpose of avoiding the ill effects of frost and possibly of preparing for future developments that may seem probable. A thickness for a concrete slab in excess of 8 in. should

be determined upon only after thoroughly considering the possibilities of meeting the necessities of the case by other means, such as improving the sub-grade.

Although the distribution by the road crust or pavement of the traffic stresses through it to the foundation is not at present within the possibilities of calculation in the case of all types of pavements, progress has been, and is being, made along these lines. Logical and fairly accurate formulas have been developed in the case of broken stone road crusts, and studies productive of some results have been made in the case of some other surfacings. Also, the bearing power of soils of different kinds has been given considerable study. These studies should be encouraged and carried on, so that the thickness of the road crust or pavement for any type may be rationally determined.

Uniformity of thickness for the surfacing is made for the purpose of conducing toward uniformity of wear. Variations in the thickness of such surfacings as sheet-asphalt, for instance, invariably result in non-uniformity of wear, with a resulting increase of expense for satisfactory maintenance; and it is believed that the same cause is responsible for many of the difficulties experienced in the maintenance of block pavements of various types. The development of depressions in the surfaces of these pavements and the deterioration of areas of the pavement seem to be explained by the irregularities in the thickness of the surfacing or of the cushion or foundation under the surfacing over such areas, which irregularities produce unequal settlement or decided differences in rigidity.

The thickness of the pavement or surfacing, of course, will be dependent largely on its type. Approved practice establishes the limits shown in Table 1 for the extremes of thickness for the various layers of the pavement or road crust.

TABLE 1.

Kind of roadway.	Thickness of artificial foundation* (ordinary), in inches.	Thickness of sand cushion or binder course, in inches.	Thickness of wearing course, in inches.
Asphalt block	5 to 8	....	2 to 3½
Bituminous surfaces	4 to 8	....	¼ to ½
Bituminous concrete	3 to 8	....	1½ to 3
Bituminous macadam	3 to 8	....	2 to 3
Brick	4 to 8	¾ to 1½	3 to 4
Broken stone	3 to 8	....	2 to 3
Cement-concrete { One course	....	....	5 to 8
Cement-concrete { Two course	4 to 8	....	2
Gravel	4 to 8	....	2 to 4
Sheet-asphalt	5 to 8	1 to 1½	1½ to 2
Stone block	5 to 12	1 to 2	2½ to 5
Wood block	5 to 8	0 to ½	3½ to 4

* Not including extraordinary provisions such as V-drains or "sub-base" courses.

Drainage.—The use of any form of pavement or road crust, whether bituminous or non-bituminous, does not relieve the necessity for proper drainage in every case. It is not only necessary to provide for such under-drainage as will place and keep the sub-grade in a condition satisfactorily free from moisture and in a state of suitable efficiency, but it is also necessary to provide and to preserve economically such provisions for surface drainage as will, with the provisions for under-drainage, insure these results fairly permanently. Storm-water coming to the roadway must be carried quickly and rapidly away from it by automatic arrangements to the natural watercourses where it can be finally disposed of. The arrangements referred to and so made, such as inlets, ditches, gutters, and culverts, should be designed and placed so as give the least possible offense to the users of the roadway and the abuttors, and yet be built so as to preserve their integrity and efficiency with the least need for attention and expense, under even the most persistently adverse natural conditions. A proper longitudinal grade for ditches and gutters is particularly important, in order that the ill effects of standing water may be avoided. A proper cross-section for ditches is also important, in order that the water may not become obstructed by the sliding in of the sides.

The under-drainage of the roadbed, where a cement-concrete roadway or an artificial foundation of cement-concrete is to be provided, should be at least as good as that which would be required in most cases of other surfaces, because the rigidity of the cement-concrete slab does not permit it to adapt itself—as is the case, for instance, with such a surface as macadam—without injury to changes in the sub-grade resulting from defective drainage.

As related to drainage, the matter of the crown of the roadway is particularly important. The ideal roadway surface would be flat in cross-section were it not for the necessity for the removal of surface water to the channels where it must be most conveniently carried along. Crowning the roadway tends to concentrate the traffic on the ridge, where it is then most comfortable for the travelers, and the amount of crown which will result in this concentration on the ridge varies with the type of pavement. Also, the rate of crown necessary for the proper removal of storm-water to the gutters or ditches varies with the type and with the provisions to be made for the cleaning and the upkeep of the roadway surface. In the general practice, the amount of crown for the shoulders of an uncurbed roadway has usually been a cross-slope of 1 in. per ft., the shoulders being of the natural earthy material, and this rate is to be recommended for shoulders, except in special cases.

With pavements inclined to be slippery under certain conditions the crown should be reduced to the lowest possible minimum consistent with surface drainage; and, where the longitudinal grade is sufficient

to allow the water to run off freely, the crown should be very flat—not exceeding 3 in. in a roadway width of 30 ft.

For the various roadway surfacings, the practice generally observed and to be recommended is stated in Table 2.

TABLE 2.

Kind of roadway.	Maximum.	Minimum.
Asphalt block	¼ inch to the foot.	⅛ inch to the foot.
Bituminous surfaces	½ " " "	¼ " " "
Bituminous concrete	½ " " "	¼ " " "
Bituminous macadam	½ " " "	¼ " " "
Brick pavements	⅜ " " "	⅛ " " "
Broken stone	¾ " " "	½ " " "
Cement-concrete	⅜ " " "	¼ " " "
Gravel	1 " " "	½ " " "
Sheet-asphalt	¼ " " "	⅛ " " "
Stone block	½ " " "	¼ " " "
Wood block	¼ " " "	⅛ " " "

Concave pavements of cement-concrete, vitrified block, or stone block, may frequently be found advantageous for alleys, and, in such cases, the same rates of slopes in cross-section as those given in Table 2 should govern.

Sub-Grade.—The use of any form of pavement or road crust does not relieve the necessity for the construction of a well-drained, thoroughly compacted, homogeneous, and stable sub-grade in every case. Indeed, such improvement of the highway generally attracts heavier traffic, and thus increases the stresses in the sub-grade. Even when an artificial foundation is to be constructed on the sub-grade, proper care should be taken in its preparation in order that the greatest economy may be had in the design and expense for the artificial foundation, and, generally speaking, at least, the higher the type and the more expensive the artificial foundation, the greater care should be had to develop to the utmost the possibilities of the sub-grade. Uniformity in the composition and compaction, as well as evenness of its surface, is far more important than has been generally considered necessary, and permanence of all the desirable qualities in the sub-grade is equally important.

When an artificial foundation or a cement-concrete pavement is used, the sub-grade should be as carefully prepared, rolled, and compacted, as for any other roadway surfacing, and should be made to conform accurately to the proper lines and grades. If necessary, the sustaining powers of the natural foundation should be reinforced and the strains in it thereby further distributed by the interposition between it and the cement-concrete slab of an artificial foundation consisting of a layer, or layers, of sand, gravel, broken stone, or similar material. Reliance should not be placed on the concrete, if used, for bridging soft, spongy, or unyielding spots; all vegetable or

perishable matter should be removed from the sub-grade, and such other material substituted as will insure a thoroughly compacted and homogeneous sub-grade for the concrete. It may be necessary, for proper compaction, to use water in connection with rolling, and, in any event, cement-concrete should not be deposited on a dry, absorbent sub-grade.

Every precaution should be taken to prevent a disturbance of the sub-grade after it is completed and until the next layer is deposited.

When a cement-concrete or other rigid pavement is to be constructed over an old surfacing composed of gravel or broken stone macadam, the old pavement should be loosened, spread to the full width of the new pavement, and then thoroughly compacted, filling the interstices with fine material and re-rolling until a dense sub-grade is obtained.

Artificial Foundations.—Where the character of the traffic justifies the use of an artificial surfacing, it also demands a correspondingly strong foundation. Whether or not an artificial foundation shall be supplied will depend on the local conditions, but, in the selection of the materials and the methods of construction of the artificial foundation, every consideration should first be given to the possibilities for securing the greatest efficiency from the natural foundation. Economy in reference to the roadway will be had from the proper choice of the various materials available for artificial foundations, such as sand, gravel, broken stone, and concrete.

Owing to the inherent lack of elasticity in brick or cement-concrete pavements, it is especially necessary that the surface of the wearing course of such pavements shall be built and remain smooth and true to contour, for the sake of ease of traction, comfortable riding, and integrity of the surface, particularly where a cement filler for the joints is used. Special care, therefore, should be taken in these cases to provide foundations of ample strength and permanence to this end. Stone block pavements should generally, and wood block pavements should always, be laid on a cement-concrete foundation, but, in case of temporary paving with stone blocks, any type of stable foundation may be used.

Local conditions, occasionally, may justify the omission of an artificial foundation for brick pavements where the natural foundation material can be satisfactorily prepared and a reasonable permanence then expected from it, where a relatively low first cost of the brick prevails and where light traffic only is to be expected.

Any artificial foundation should be of a substantial thickness and be properly consolidated; and should be rendered so compact that further settlement or displacement will be avoided to the utmost.

The most usual proportions for a cement-concrete foundation have been one part cement, three parts fine aggregate, and six parts coarse

aggregate. This standard, however, is empirical rather than scientific, and a more rational proportion should be developed according to the needs and facilities of each case. Sometimes it may be desirable to increase the mass at the expense of unit strength, or to increase the mass for the sake of economy in the more expensive material. In mixing, placing, and completing a cement-concrete foundation, the principles expressed under the head of "Cement-Concrete Pavements" apply, and reference thereto should be had.

Materials.—Having determined the characteristics desirable for the materials to be used in any highway work, their description in the specifications should be concise, clear, and exact. Although, in some cases, it may not be possible to designate precisely the characteristics desired, it will be possible to specify that these qualities shall lie between certain limits, thus giving a reasonable tolerance to the determination of the quality by test as well as avoiding uncertainty as to whether or not a quality in this respect of a material offered is suitable. The description of a material by means of a trade name is permissible only in most unusual cases, and such a description as "equal to" another similar material should never be used. Qualities of a material or methods of its use should not be left "to the satisfaction of the engineer" or "as determined by, or in the opinion of, the engineer."

Specific tests and such description of the methods of performing each test as will leave no room for doubt as to whether the results of the tests come within the limits of tolerance should always be expressed, either in detail or by reference to the standards of some reputable authorities. The tests and methods of performing them, to be found in detail in Appendices B and C, are recommended for this purpose.

Whenever comprehensive specifications are to be prepared so as to admit a variety of types of bituminous materials, separate specifications, as may be necessary, should be written for each type.

Joints.—For the ordinary joints in block pavements, the materials and methods of filling should be selected so as to produce, not only a surface which will retain to the utmost its imperviousness and the stability of the blocks themselves in place, but, also, they should, as far as practicable, conduce toward evenness of wear of the surface of the pavement. If the blocks are resistant to abrasion, but are inclined to round off at the edges of the upper surface under traffic, such filling of the joints is desirable as will reduce rounding off at the joints. On steep grades, where some roughness of surface may be desirable for the sake of affording better foothold for animals, some openness at the top of the joint is desirable, and the softer joint fillers, less resistant to wear, may be preferred.

Joint fillers naturally are divided into two main classes—the hard cement-mortar filler and the soft bituminous filler. As it is desirable

to secure water-proof pavements, sand alone should never be used as a joint filler. Though the use of sand joints may occasionally appear to be justified in the interests of economy, it will generally be found unwise to use such joints when some relatively slight additional first cost will result in appreciably prolonging the life of the pavement. A bituminous filler may be preferred to a cement-grout filler on account of the lower cost of street opening repairs, the better foothold provided for horses, and the securing of a more elastic and less noisy pavement.

Cement-mortar joints when properly made will conduce to integrity of the surface. The proportions of sand, cement, and water will be affected by local conditions. To insure the best results, a 1:1 mix of sand and cement is recommended. Great care is necessary in mixing and applying the mortar or grout. Uniformity in the cement grout, and especially skill and care in its application, are essential to success. To insure uniformity, there should be a constant agitation of the mix, up to the moment of its application, and no more water than is necessary for proper fluidity should be used. Ample time should always be allowed for the grout to set thoroughly before the traffic is admitted to the roadway.

With bituminous joint fillers, care must be taken to select materials which will not be too brittle in cold weather and so chip out from the joints under traffic, and which will not be so soft in hot weather as to flow out of the joints between the blocks. It is believed, although not yet generally admitted as having been actually proved by experience, that the use of a bituminous mastic for joint filling would be an improvement over the customary practice of using bituminous material alone for this purpose. One of the great difficulties with bituminous fillers of any kind will be that of properly filling the joints between the blocks, and great care must be taken to insure this result.

Expansion-Contraction Joints.—Joints at intervals across certain types of pavements, such as brick and cement-concrete, as well as along the curbs, have been used in order to compensate for a more or less unavoidable movement of the pavement slab, which takes place under different conditions of moisture, or temperature of the air. In cases where expansion-contraction joints across the roadway at intervals are decided on, the Committee recommends the use of bituminous material and the abandonment of all forms of the so-called "armored" joints, because of the smaller interruption to the homogeneity of the roadway surface thus secured.

Cushion Courses.—The function of a cushion between the brick or block on an artificial foundation of cement-concrete is to allow for irregularities in the surface of the concrete and in the depths of the brick or block, and to give resiliency to the wearing course. If the surface of the concrete foundation is made true to the adopted cross-section, and as the variation in depths of the brick or block decreases,

the thickness of a sand cushion may be correspondingly decreased. The thickness should never be greater than that necessary to compensate for the unevennesses referred to plus such a thickness in the layer as will enable the latter to perform satisfactorily its function as a cushion, that is to say, about ½ in. for this latter addition. If the surface of the concrete is truly parallel with the finished pavement, and if the variation in the depths of the bricks or blocks does not exceed ⅛ in., the thickness of a sand cushion can safely be reduced to ¾ in. The material which has generally been used for the cushion course is sand, but engineers, recently, have been considering the advisability of using Portland cement mortar instead, the mortar being spread either on the surface of the concrete foundation after the latter has been completed, or being laid as a part of the foundation itself. The objection made by some to mortar is that it makes too solid a base for the blocks and gives no resiliency. This at present seems to be a moot question. The substitution of the mortar results in a monolithic structure, perhaps of greater strength, but less capable of absorbing shock without injury than is the case where a sand or bituminous cushion is introduced between the wearing surface and the foundation, especially where the joints are filled with a bituminous filler.

The sand for a cushion course may be slightly more loamy than that permissible for safety in mortar or cement concrete work. This excess loaminess may be advantageous in offering the sand a greater ability to resist displacement; but it should not be so loamy or fine as to prevent proper spreading and compaction, or to afford such a degree of capillarity as will result in frost action.

A bituminous mixture composed of sand, stone screenings, or possibly wood fiber, and a bituminous cement would probably be more satisfactory for many reasons than either the mortar course or the sand alone, and such a bituminous mixture needs to be only from ¼ to ½ in. in thickness, provided the surface of the concrete foundation is made sufficiently smooth and regular in contour.

Finishing of Surface.—An objectionable slipperiness of many pavements may be decreased or prevented by proper precautions during construction or by proper treatment thereafter. The length of time that a finished pavement should be closed to traffic in order to season properly before use varies from a few hours to several days, dependent on the character of the material and methods used and on climatic and other local conditions. Pavements in which Portland cement is used for filling the joints or in the mass of the surfacing itself should seldom, if ever, be closed for less than 2 weeks after completion. Although the plans and specifications usually call for the surface to be finished to definite cross-sections, grades, or contours shown on the

plans, questions frequently arise under contracts as to the importance of variations in exactness of compliance in the finished surface.

The Committee has had a number of observations made, from the results of which it is convinced that, in a newly completed pavement, the variations from a straight-edge or template, 8 ft. in length, should not exceed ⅛ in. for asphalt block, bituminous concrete, brick, cement-concrete, sheet-asphalt, and wood block pavements, and ¼ in. for broken stone roads, and bituminous macadam and stone block pavements.

Manhole Covers and Street-Car Tracks in Roadway Surfaces.—Uniformity in the roadway surface being essential for minimum wear and expense of maintenance, anything, such as manhole covers or street-car tracks, introducing an element of non-uniformity into the surface, should be counteracted as far as possible whenever surfaces of different degrees of hardness adjoin in a pavement. The traffic coming from the harder to the softer surface naturally causes abnormal wear on the latter.

Practically all manhole covers are laid on rigid masonry, so that, unless some special treatment is given to the pavement, there is apt to be undue wear adjacent to such structures. Where the pavement is of stone block, wood block, or brick, the difference in hardness between them and the manhole heads is not so great as with other paving materials, so that, when these pavements are used, they can be laid flush with the manhole covers, provided extra care is taken with the foundation immediately adjacent to the heads, to prevent any possible settlement. In the case of macadam or bituminous pavements, a different treatment should be used. In such cases the pavement should be laid about ⅛ in. above the manhole head. This will prevent the abnormal wear caused by the pounding action of the wheels of vehicles, which would occur if the pavement surface was any distance below that of the manhole head, a condition which must exist if the pavement is laid level with such heads, as it is not possible to compact the pavement so that it will not compress under the traffic, forming depressions before those actually caused by wear begin.

In the case of car tracks in streets, modern construction is such that the tracks are nearly rigid, although this condition does not exist in all cities at present. Where it does exist, however, the pavement should be laid, as in the case of manhole heads, somewhat above the level of the rail. The Committee, however, does not believe that in any case a bituminous pavement should be laid between the tracks or between the rails of the tracks. When car tracks are laid in roads the construction is not generally as nearly rigid as in streets, and the rails are usually of the T form. The tracks in such cases are often laid at the side of the road, rather than in the center, as is customary in streets. In the case of macadam or bituminous roadways, and when the rails are in the center, it would be advisable to lay stone blocks or

brick for a width of at least 18 in. adjacent to the rails; when the rails are at the side and the railroad area does not form a part of the roadway proper, loose broken stone or gravel may be substituted for the stone blocks or brick.

Asphalt Block Pavements.*

General.—Specifications for asphalt block pavement should cover thoroughly the several components of the bituminous concrete used, the manufacture of the blocks, the blocks *per se*, and the details of construction of the pavement.

Materials.—Experience has demonstrated that the blocks should be composed of asphalt cement, crushed trap rock or equally hard and tough material, and mineral dust. All particles of the trap rock should pass a ½-in. screen, and the mineral dust or filler should consist of powdered limestone. The bitumen content of the blocks should be between 6.5 and 8.5%, depending on the grading of the mineral aggregate and the method of manufacture. The specifications should contain specific requirements with reference to the asphalt cement, filler, and the grading of the mineral aggregate, which latter should be similar to the following:

Passing 200-mesh sieve	20 to 35 per cent.
Passing 80-mesh sieve and retained on 200-mesh sieve	7 " 15 " "
Passing 20-mesh sieve and retained on 80-mesh sieve	12 " 30 " "
Passing ½-in. screen and retained on 20-mesh sieve	30 " 50 " "
Retained on ½-in. screen	0 " "

The specifications should also cover the specific gravity of dry blocks, which should not be less than 2.45 at 25° cent. (77° Fahr.) and the percentage of absorption of water of the blocks, after being dried for 24 hours at a temperature of 65° cent. (149° Fahr.), should not be more than 1% after immersion in water for 7 days. The blocks should be about 5 in. in width and 12 in. in length, and 2, 2½, or 3 in. in depth, depending on traffic conditions.

Construction.—The blocks should be laid on a fresh ½-in. mortar bed which covers a cement-concrete foundation. After being laid, the blocks should be covered with a thin layer of clean, dry, fine, sharp sand which should be thoroughly swept into the joints until they are filled.

Bituminous Concrete Pavements.*

Classification.—The principles to be covered in drafting specifications for bituminous concrete pavements will be grouped under the

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

three classes into which these pavements generally may be divided These classes are described as follows:

Class *A*.—A bituminous concrete pavement having a mineral aggregate composed of one product of a crushing or screening plant;

Class *B*.—A bituminous concrete pavement having a mineral aggregate composed of a certain number of parts by weight or volume of one product of a crushing or screening plant and a certain number of parts by weight or volume of sand, broken stone screenings, or similar material, with or without a filler;

Class *C*.—A bituminous concrete pavement having a predetermined, mechanically graded aggregate composed of broken stone, broken slag, gravel, or shell, with or without sand, Portland cement, fine inert material, or combinations thereof.

Bituminous Concrete Pavements, Class *A*.

Mineral Aggregate.—Broken stone, because of the satisfactory bond secured, should be used wherever possible, although bituminous concretes constructed with gravel have proved satisfactory for light traffic where great care has been taken in the selection of the gravel and in the construction of the pavement.

Broken stone should be clean, rough-surfaced, sharp-angled, of compact texture, and uniform grain. If the pavement is to be subjected to medium or heavy traffic, the broken stone used for the construction of the wearing course should show a loss or abrasion of not more than 3.5%, and its toughness should not be less than 13.

Especial care is required in drafting the specifications covering the broken stone or gravel to be used. An excess of large or small sized stone or gravel should be avoided. Practice has demonstrated that a mineral aggregate composed of those materials which will comply with the following mechanical analysis, using laboratory screens having circular openings, will produce satisfactory results: All the material shall pass a 1¼-in. screen; not more than 10% nor less than 1% shall be retained on a 1-in. screen; not more than 10% nor less than 3% shall pass a ½-in. screen.

Bituminous Cements.—Experience has demonstrated that the most efficacious bituminous concrete pavements of Class *A* are constructed by using suitable asphalt cements or refined tars in the mix, and asphalt cements for seal coats. Satisfactory results will be secured when tar cements are used for seal coats, but the surface must be re-treated more frequently than when asphalt cements are used.

Heating Aggregates and Bituminous Cements.—Although satisfactory pavements have been constructed using unheated mineral ag-

gregates and suitable bituminous cements, service tests demonstrate that the best results are secured by using for the mineral aggregate broken stone or gravel which is heated until thoroughly dry to between 66° cent. (150° Fahr.) and 121° cent. (250° Fahr.). If revolving dryers in which the flame is permitted to come in contact with the aggregate are used, great care should be taken to ensure uniformity of heating and so avoid the danger of burning the aggregate.

In order to obtain a fluidity of the bituminous material which should be sufficient to ensure a proper coating of the mineral particles in cases where a heated aggregate is used, and also to prevent injury to the bituminous material, the asphalt cements should be heated to a temperature between 135° cent. (275° Fahr.) and 177° cent. (350° Fahr.), and refined tars to a temperature between 93° cent. (200° Fahr.) and 135° cent. (275° Fahr.).

Mixing.—The quantity of bituminous cement to be used in the mix will depend on the kind of broken stone or gravel and bituminous cement, the character of the aggregate, the climatic conditions, etc. For the aggregate heretofore mentioned, the bituminous concrete mixture should contain between 5 and 8% by weight of bitumen.

The bituminous concrete should be mixed in mixers designed and operated so as to produce and discharge a thoroughly coated and uniform mixture of non-segregated aggregate and bituminous cement. Except on small contracts and for repair work, mixers which provide for the heating of the aggregate by the use of a flame in the mixing chamber should not be used, on account of the danger of burning the aggregate or the bituminous cement.

Laying.—To ensure ease of manipulation and the proper compaction of the bituminous concrete, the mixture as delivered on the roadway should have a temperature of not less than 66° cent. (150° Fahr.). Experience has demonstrated that a thickness, after rolling, of 2 in. of bituminous concrete is economical and efficacious. In order to secure an even surface and adequate compaction by a thorough interlocking of the particles of the aggregate, a tandem roller weighing between 10 and 12 tons should be used.

Seal Coat.—A seal coat should always be used on this type of bituminous concrete, as maintenance charges and annual cost will be reduced materially thereby. The seal coat should consist of from ½ to 1 gal. per sq. yd. of bituminous cement, uniformly distributed, preferably by the use of a hand-drawn distributor followed by a squeegee. The bituminous cement should be covered with an application of dry stone chips, which should be rolled.

Seasonal Limitations.—Bituminous concrete of this type should not be mixed or laid when the air temperature in the shade is lower

than 10° cent. (50° Fahr.), as otherwise it is difficult, under average conditions, to secure an even and well compacted wearing course.

Bituminous Concrete Pavements, Class *B*.

Specifications for pavements of this class have generally stipulated that so many parts of broken stone or gravel and so many parts of sand or other fine material are to be mixed with a certain quantity of bituminous cement. By the use of this specification, and with unusual supervision, it is practicable to secure a fairly well-graded aggregate, but in most cases the mixture will be found to contain an excess of broken stone, with insufficient fine material to fill the voids therein, and in other cases it will contain an excess of sand in which the broken stone is held as isolated particles.

Bituminous Concrete Pavements, Class *C*.

This type includes the so-called "Topeka" mixture, and several kinds of patented pavements.

Topeka Bituminous Concrete Pavement.—If the Topeka pavement specification embodies the grading, as contained in the decree of 1910, namely,

Bitumen, from 7 to 11 per cent.

Mineral aggregate, passing 200-mesh screen, from 5 to 11 per cent.

Mineral aggregate, passing 40-mesh screen, from 18 to 30 per cent.

Mineral aggregate, passing 10-mesh screen, from 25 to 35 per cent.

Mineral aggregate, passing 4-mesh screen, from 8 to 22 per cent.

Mineral aggregate, passing 2-mesh screen, less than 10 per cent.,

special provisions should be made in the specifications covering the broken stone and sand to be used, in order to secure satisfactory grading. Otherwise, the principles stated under "Bituminous Concrete Pavements, Class *A*", should be followed, except that a seal coat is not considered necessary under many conditions where this type of pavement is used.

Patented Bituminous Concrete Pavements.—In cases where patented bituminous concrete pavements of Class *B* or Class *C* are used, the same fundamental principles observed under "Bituminous Concrete Pavements, Class *A*" should be followed, especially in the case of covering in detail the composition and grading of the mineral aggregate, and the physical and chemical properties of the bituminous cements used.

Bituminous Macadam Pavements.*

Materials.—The broken stone should be of a quality equal to that prescribed for broken stone roads, and should have the same characteristics. The bituminous materials may be of asphalt or refined tar.

Construction.—The principles relating to thickness applicable to a broken stone road are likewise applicable to bituminous macadam pavements, and thorough rolling, including the rolling of the upper course, both before and after the application of the bituminous material, is also necessary. As it is desired to bind only the upper course with bituminous material, it is necessary, in order to prevent waste by penetration, that there should be no appreciable voids in the next lower course. It is not necessary, however, to flush the filler or binder in this course to the same extent as is necessary in binding the top course of a water-bound road, and it is absolutely essential that no binder should cover the stones of the lower course when the top course is spread.

The quantity of bituminous material used should be only sufficient to penetrate through the upper course. The quantity per square yard cannot be prescribed absolutely, depending in some degree on the hardness and size of stone used, but, in general, the application of 1 gal. or less to the square yard for each inch in thickness of the finished upper course is adequate.

The use of a pressure distributor in applying the bituminous material is essential, and the distributor should be of such type that absolutely uniform application may be accomplished, and that no ruts are formed in the surface by the wheels supporting the distributor.

The bituminous material should be applied at such a temperature that it will flow freely, and, to insure proper penetration, the stone should be dry and clean, and the air temperature should not be lower than 10° cent. (50° Fahr.) during application.

In order to secure a proper surface, the covering material should preferably consist of the crusher product passing over a ½-in. screen and through a ¾-in. screen. Finer material, however, may be used for covering if a slippery surface is not objectionable, but the use of material passing through a 10-mesh sieve should be avoided.

The use of a bituminous material by no means justifies any lack of care in the ordinary details to be followed, but rather increases the need for thoroughness and skilled supervision. The main principles underlying good construction in water-bound roads remain in full force when such roads are treated with bituminous material. Whatever method may be used in any case, it is as essential, in bituminous work as in water-bound construction, that a suitable quality of road metal be used.

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

Bituminous Surface Treatments.*

Description.—The proper treatment of a broken stone, gravel, shell, or slag roadway with bituminous material, for the purpose of eliminating the so-called dust nuisance, will at the same time render even the best of such roadways more efficient for sustaining traffic, and such treatment with bituminous materials is usually preferable and more economical than sprinkling with water, or the use of hygroscopic salts.

Bituminous Material.—Either refined tar, cut-back asphalt, or asphaltic oil may be used for surface treatment. If the surface to be treated is a gravel, broken stone, slag, or other porous and non-bituminous crust, practice has proven that bituminous material of such consistency that it can be applied at a temperature below 52° cent. (125° Fahr.) is preferable to heavier material, and that on any crust the application of a quantity in excess of ½ gal. per sq. yd. is inadvisable. It is advisable to apply the material in quantities not exceeding ¼ gal. per sq. yd. at a time. Heavier material in less quantity, however, should be used on bituminous roadways; otherwise there will be a tendency toward an objectionable softening of the material previously used in the construction of the roadway.

Construction.—The surface to which a bituminous treatment is to be applied should be dry, compact, and free from depressions and dust. On a broken stone road, the application should be made on the exposed stone surface of the upper course, such exposed surface being obtained by thoroughly removing with brooms or sweepers the binding material or dust that may have been applied or accumulated thereon. The bituminous material, in all cases, should be applied by a pressure distributor designed so that the material will be spread uniformly and with a pressure of not less than 20 or more than 75 lb. per sq. in.

The application, in all cases, should be carried over the outside edges of the rolled metal and on the shoulder far enough to protect the edges of the metaled surface. The material should be applied only after the surface has been thoroughly compacted by traffic or otherwise.

After the bituminous material is applied it should be covered with the toughest grit obtainable, preferably of a size that will pass through a screen having openings of not less than ⅜ in. nor greater than ⅝ in., just enough of such material being used to cover the bituminous material. It is advantageous, but not entirely necessary, to roll with a steam roller after the application of the grit.

Brick Pavements.†

Cushion Course.—In the case of brick pavements, the brick being of uniform size, sufficient resiliency will be secured by the use of a

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

† For discussion of rates of grade, crown, artificial foundation, joints, etc., see "General Conclusions."

sand cushion 1 in., or even slightly less, in depth, provided the depth is uniform and the surface of the concrete foundation is truly parallel with the finished pavement; and if the variation in the depth of the brick does not exceed ⅛ in., the thickness of the sand cushion can safely be reduced to ¾ in. In many brick pavements recently laid the cushion course has been dispensed with entirely, the brick having been bedded in cement mortar spread over the concrete foundation. This results in a monolithic structure less capable of absorbing shock than is the case where a sand or bituminous cushion is introduced between the wearing surface and the foundation. A cushion course composed of sand or stone chips and a bituminous cement about ½ in. in thickness may be substituted for sand or cement mortar, provided the surface of the concrete foundation is made sufficiently smooth and regular in contour.

Materials.—The quality of the brick should be determined by physical tests. The method of making the rattler test adopted by the American Society for Testing Materials is approved by the Committee. This test will indicate the toughness and resistance to wear from shock and abrasion. Uniformity in the rate of wear is so important that it properly may be a controlling consideration, even at the expense of a moderate increase in the rate of wear. In size and shape it is desirable to conform to accepted standards in order that repairs and renewals may more readily be made. Uniformity in size is especially important, and variations in depth should be kept within the narrowest limits.

Construction.—The bricks should be laid in straight courses at right angles to the axis of the roadway, although at intersections they may advantageously be laid in diagonal courses arranged so that traffic turning any of the corners will move across the bricks and not along the continuous joints. They should be laid so that the joints will be uniform in width and of sufficient width only to permit a filler to reach the bottom of the joints. Lug bricks have the advantage of insuring such uniform width with ordinary care in laying. If a sand cushion is used, great care should be taken to avoid any disturbance of the surface of the cushion after it has been brought to true grade by using a template. If bedded in a mortar or bituminous cushion, the brick should be bedded so that the surface will be as true as possible. In all cases the brick after being laid should be brought to a true and even surface by the use of a roller.

Broken Stone Roads.*

Materials.—All broken stone should be clean, rough-surfaced, angular, of compact texture, and of uniform grain. It should preferably be of such quality that, using standard laboratory tests, it will show a

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

percentage of wear not greater than 5 (French coefficient of wear not less than 8), and a toughness of not less than 6.

The broken stone should be separated into component sizes by passing the product of the crusher over rotary screens having circular openings, the different sizes of stone being collected in separate bins. The separation of sizes is governed somewhat by the thickness at which the crust is to be laid. If a 6-in. surface is to be laid, the maximum size should not exceed that of stone passing a 3-in. screen, whereas, if the pavement is to be 7 in. or more in thickness, a 3½-in. screen is permissible. In all cases the stone should be spread and rolled in two or more courses, the largest size generally being used in the lower course and the smaller sizes in the upper course or courses, each course being spread in such manner that there will be uniformity in its density. Such uniformity is accomplished by spreading from piles dumped on boards or alongside the road, or by automatic spreading wagons.

The product passing through a ⅜ to ¾-in. screen should be used as a binder, and on two-course work should be applied on the top course only, whereas, on the three-course work, the second course should be lightly covered with this material.

Products of broken stone obtained from portable and stationary stone-crushing and screening plants, though complying with a given specification of the type now ordinarily used, vary to a considerable extent, due to variations in the plant and its operation, such as kind of rock, type and method of operation of crushing plants, methods of separation of products, differences in lengths and diameters of sections of rotary and shaker screens, and differences in inclination and rate of operation of screens.

The necessity for more carefully drawn specifications covering the sizes of the particles of which a given product of a stone-crushing and screening plant is composed, is illustrated by Table 3, mechanical analyses of two products obtained from the same plant, both of which products passed over a section of a rotary screen having circular holes 1¼ in., and through a section of a rotary screen having circular holes 2¼ in. in diameter.

TABLE 3.

	Sample "A." Percentage.	Sample "B." Percentage.
Passing ¼-in. screen	0.3	0.2
" ½ " " and retained on ¼-in. screen.	0.4	1.1
" ¾ " " " " " ½ " "	2.2	12.6
" 1 " " " " " ¾ " "	8.0	37.5
" 1¼ " " " " " 1 " "	29.1	40.9
" 1½ " " " " " 1¼ " "	27.1	7.7
" 2 " " " " " 1½ " "	32.9	0.0
	100.0	100.0

It is obvious that, for many forms of construction, in order to secure successful results, greater care must be used in the writing of specifications for products of broken stone. The Committee recommends the general use, as soon as practicable, of the "Standard Form of Specifications for Certain Commercial Grades of Broken Stone", as recommended by Committee D-4, of the American Society for Testing Materials, in its 1916 Report:

The broken stone shall consist of one product of the operation of a stone-crushing and screening plant, without re-combining or mixing, and shall conform to the following mechanical analysis, using laboratory screens:

Passing .... in. screen (having smallest holes selected) from .... to .... per cent.

Passing .... in. screen (having next to largest holes selected) from .... to .... per cent.

Passing .... in. screen (having largest holes selected) from .... to .... per cent.

In this form of specification an attempt is made to cover in the mechanical analysis only the limits of the smallest and largest particles. No attempt is made to secure a carefully graded aggregate, but simply a product suitable for the type of road or pavement in question.

An engineer should base the selection of screens, to be used in the specification for a given product of broken stone, on the results of mechanical analyses of many similar products obtained from portable and stationary crushing and screening plants which supply the locality in which the specification is to be used.

Construction.—Each course of a broken stone road should be thoroughly rolled with a roller weighing from 10 to 15 tons, the rolling being done first along the sides and gradually approaching the center, and being continued until there is no movement of the stone ahead of the wheels of the roller.

The binder should be used on the top course in such quantity that, after alternate spreading of binder and watering, with continuous rolling, the voids become so filled as to result in a wave of grout being pushed along the surface by the front wheel of the roller.

After the completion and binding of the top course, a thin layer of screenings or stone dust should be applied to the surface in sufficient quantity to cover it evenly.

Cement-Concrete Pavements.*

General.—A thickness of from 5 to 8 in., as stated in the general principles, may ordinarily be considered sufficient for a concrete slab, and, if it seems advisable, from motives of economy, the thickness may

* For discussion of rates of grade, crown, artificial foundation, joints, etc., see "General Conclusions."

be diminished from the center of the slab to the edges. Special conditions may call for variations, even outside of the limits given. The character and the drainage of the sub-grade, its probable stability as a foundation, as well as the nature and amount of traffic, are some of the factors which enter into the rational determination of the thickness of the slab.

Materials.—The cement should be tested by the methods recommended by the Special Committee on Uniform Tests for Cement* of the American Society of Civil Engineers, and should meet the requirements adopted by the American Society for Testing Materials, as printed in the 1916 Year Book of that Society.

The Committee wishes to emphasize the importance of the aggregate in making up the concrete structure. Fine aggregate may be considered as gravel, sand, or screenings from hard, durable rock, graded so that, when dry, it will pass a screen having ¼-in. circular openings. The best fine aggregate is in general that which is graded fairly uniformly from the ¼-in. size mentioned above, downward, but not more than 5% should be of such fineness that it will pass a sieve having 100 meshes per lin. in. A preponderance of the coarser particles rather than the finer is desirable, and, in any case, less than 3% of the material should pass the 200-mesh sieve. Standard briquettes, made of samples of fine aggregate and tested at the usual periods of 7 and 28 days, should show a strength at least equal to similar briquettes made of the same cement and three parts of standard Ottawa sand. Coarse aggregate, in general, should not be larger than will pass a screen with 2-in. circular openings, ranging down fairly uniformly from this size to that retained on a ¼-in. screen. The loss of such material, as determined by the abrasion test, should not be more than 5 per cent.

A denser and more uniform concrete may be made by screening the material, both fine and coarse aggregates, into different sizes and re-combining these different sizes in such a way as will give the densest mixture, that is, the smallest percentage of voids, when dry. Although this adds somewhat to the expense, the resulting composition will generally be found enough better to make up for the additional expenditure. Furthermore, in proportioning the ingredients of the concrete, a mixture based on mechanical analyses is in general to be preferred to the arbitrary rule of a 1 : 2 : 4 or 1 : 3 : 5 mix. The slight increase in time and expense which is incurred by determining the voids and combining the various sizes to get the greatest density is more than repaid by the strength and density of the resulting mixture. Similar care in the determination of the proper quantity of water is also to be desired in order that the concrete may have a uniform consistency as it is deposited in place. Though the mixture should be rather wet, especially if it is deposited without tamping, it should still be stiff

* *Transactions*, Am. Soc. C. E., Vol. LXXV, 1912, p. 665.

enough to hold its shape when struck off by the template, and yet not result in segregation of the different sizes throughout the mass. Water used in mixing should be clean and free from oil, alkali, or vegetable matter.

Construction.—Forms used in cement-concrete pavements should be as carefully considered as those for any other class of structural work in concrete. It is important they be true, and free from warp, and of sufficient strength to hold the wet concrete without springing out of shape. Particular care should be taken to keep the forms tight, so that leakage through the sides, which might allow the cement or mortar to be carried out of the coarse aggregate along the edges of the roadway, may be effectively prevented. The concrete should be deposited rapidly on the sub-grade to the required depth and to the entire width of the pavement. It is better to have the surface of the rolled and finished sub-grade thoroughly dampened before beginning to deposit the concrete. Rolling or ramming the freshly placed concrete is desirable wherever practicable, as it not only increases the density of the resulting mass, but also tends to place the particles of the coarse aggregate on the surface, so that they interlock with each other and present a flat side to the wear of traffic.

Clean vertical joints, straight across the roadway, through the entire mass of concrete in place, should be insisted on when the work stops for a day, or if there is a stoppage of more than 30 min. in the work during the day. Special precautions should be taken to prevent freezing when work is carried on during cold weather, and it should be borne in mind that concrete sets much more slowly in cold weather than in warm weather. If, in the course of the work, the temperature reaches, say, 40° Fahr., and is falling, the operation of mixing and laying concrete should be suspended, and the newly laid surface suitably protected from frost.

In finishing the surface of the concrete, a template or a striking board should be used, which gives the true form of the finished pavement for its entire width. For the final surface finish, the use of the wooden float, operated from a suitable bridge which spans the entire pavement, is desirable, although satisfactory results may be obtained by the use of a properly operated belt about 10 in. wide. Care should be taken that the final surface of the pavement is true, both transversely and longitudinally, that is, with regard to both cross-section and grade. In all cases the surface of the finished pavement should be kept wet and protected from the sun for several days.

Earth and Sand-Clay Roads.*

Materials for Earth Roads.—The earth road should be constructed of the natural soil, from which, for a depth of 10 in. at the center to

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

5 in. at the sides of the roadway, all stones more than 3 in. in diameter and all sods, roots, and similar materials should be removed.

Construction of Earth Roads.—The soil should be mixed with plows, disk and spike-tooth harrows, or by other satisfactory means, until all the surfacing material is of uniform texture. When the earth roadway is completed its thickness should be not less than 10 in. at the center and 5 in. at the sides. The roadway should be shaped by the use of a road grader, and should be finished so as to conform to the desired cross-section, the crown of which should usually be about 1 in. to the ft. If any depressions appear during the grading of the roadway, they should be carefully filled with soil and the roadway reshaped. In order to secure satisfactory results, several months should elapse after the road is graded before it is considered complete; and such settlements and irregularities as develop should be corrected by the use of a grading machine or road drag.

Materials for Sand-Clay Roads.—Top-soil or mixtures of sand and clay should comply with the following requirements: The sand content should be at least from 70 to 80%; the sand used in sand-clay mixtures should preferably be composed of hard angular particles at least 30% of which should be retained on a 50-mesh sieve; the clay content should vary from 10 to 20% and under no circumstances should be allowed to exceed 30%; the clay, when placed on the roadway, should not contain lumps larger than 3 in. in diameter.

Construction of Natural Sand-Clay Roads.—The top-soil should be spread on a flat sub-grade to a depth of 10 or 12 in. at the center and 5 or 6 in. at the sides. After the surfacing material has been laid, the ditches should be constructed, and the material obtained therefrom should be used in constructing shoulders. It is advisable to plow and then harrow the surfacing material, as a more homogeneous layer is secured thereby. The surfacing material should not be compacted with an ordinary roller but with a sheep-foot roller, or the compaction should be obtained with the hoofs of animals and the wheels of vehicles going over the roadway. For successful results, thorough puddling is necessary, and this can practically only be secured through the medium of rains during the period of compaction. During compaction and after each rain the surface of the roadway should be immediately reshaped and crowned. When completed, the compacted roadway should have a depth of from 7 to 8 in. in the center and 3½ to 4 in. at the edges, and have a crown of about ½ in. per foot.

Construction of Sand-Clay Roads on Clay Sub-Soil.—Although there are several methods of construction, the following is advocated, as uniformly satisfactory results are secured by its use. The roadbed should be graded practically level, and the portions to be constructed of sand-clay should be excavated to a sufficient depth so that the earth

removed and placed on the shoulder of the road will give the right slope to the ditches. The depth of excavation should be from 1½ to 3 in., depending on the width of the road between ditches. The clay soil should then be plowed to a depth of 2 or 3 in., after which about 4 in. of sand should be spread evenly over the surface and thoroughly worked in with a disk harrow. Then 4 in. more of sand should be added and harrowed. During a rain, the sand-clay mixture should be thoroughly harrowed, and after the rain the roadway should be dragged into shape.

Construction of Sand-Clay Roads on Sand Sub-soil.—On a practically flat roadbed, a layer of clay from 2 to 4 in. in thickness should be spread evenly over the roadway. A layer of clean sand should then be spread over the clay, and the roadway thoroughly harrowed. The roadway should be shaped up with a drag, and during a rain it should be again harrowed. The roadway should be finally shaped with the use of a grading machine or drag.

GRAVEL ROADS.*

Description.—The subject of gravel roads embraces a great variety of styles of construction, from the simple expedient of surfacing the existing roadway with run-of-the-bank gravel from near-by pits to the construction of an improved highway with all the necessary drainage structures and improvements of grade attending a broken stone roadway.

Materials and Construction.—The method of treating the gravel itself may vary from the application as it is found in the pit to the separation into sizes, and even to passing the gravel through the crusher before screening. In general, we may separate gravel roadways into two classes:

First, those in which the gravel is screened and applied in the same manner as with a broken stone roadway. This may be with or without crushing the gravel.

Second, those in which the gravel is applied to the roadway in its natural state as found in the pit, with or without the addition of other material, or the natural material may be passed through the crusher before application to the roadway.

In the first case, with rounded gravel, the tendency toward dislodgment under traffic is greater than with angular broken stone, and hence, in order to reduce this tendency, the size of the pieces in the courses of the roadway must be somewhat smaller with gravel than in the case of broken stone.

In the second case, the selection of the material will be governed by the gravel available in the locality where the roadway is to be constructed. Every endeavor should be made to select a material that

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

will show, by test, its fitness as regards hardness, toughness, and cementing power. The gravel should show, on mechanical analysis, a grading of material which will contain sufficient stone of the larger size to insure stability and wearing qualities under traffic, and which will contain sufficient finer material to insure a proper bond. The best material will be composed of gravel graded so that it will have a maximum density.

Though the most available material will have a wide range as to sizes, the Committee believes that the following specification for sizes, adopted by the American Society of Municipal Improvements in 1916, may be followed safely:

"Two mixtures of gravel, sand, and clay shall be used, hereinafter designated in these specifications as No. 1 product (for top course) and No. 2 product (for middle and bottom courses).

"No. 1 product shall consist of a mixture of gravel, sand, and clay, with the proportions of the various sizes as follows: All to pass a 1½-in. screen and to have at least 60 and not more than 75 per cent. retained on a ¼-in. screen; at least 25 and not more than 75 per cent. of the total coarse aggregate (material over ¼ in. in size) to be retained on a ¾-in. screen; at least 65 and not more than 85 per cent. of the total fine aggregate (material under ¼ in. in size) to be retained on a 200-mesh sieve.

"No. 2 product shall consist of a mixture of gravel, sand, and clay, with the proportions of the various sizes as follows: All to pass a 2½-in. screen and to have at least 60 and not more than 75 per cent. retained on a ¼-in. screen; at least 25 and not more than 75 per cent. of the total coarse aggregate to be retained on a 1-in. screen; at least 65 and not more than 85 per cent. of the total fine aggregate to be retained on a 200-mesh sieve."

With gravel such a quartz, the cementation of which is low, a highly cementitious void filler is desirable, and a moderate quantity of clay or loam may be permissible.

A more generous use of gravel, especially in the surfacing of earth roads, should be encouraged. The low first cost and ease of maintenance should help materially to increase the mileage of serviceable roads in the country districts.

In drafting specifications, the refinements to be used in the methods of construction will depend on the kind and amount of traffic to be sustained, the character and quality of gravel to be secured in that particular locality, and other local conditions. Generally, however, the refinements should be carried to the point where further refinement would not be economical.

SHEET-ASPHALT PAVEMENTS.*

General.—A sheet-asphalt wearing course, consisting of predetermined graded sand, filler, and asphalt cement, should be laid to a

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

compacted thickness of not less than 1½ in. and not more than 2 in., on a binder course of bituminous concrete consisting of broken stone or broken stone and sand mixed with asphalt cement, the binder course having a compacted thickness of not less than 1 in. nor more than 1½ in.

Materials.—For heavy or medium traffic, the so-called close binder should be used, instead of the open binder, as the former possesses greater inherent stability than the latter. Specifications for the grading of open binder should be similar to those for the aggregate for Bituminous Concrete Pavements, Class *A*; and, for a close binder, similar to the following:

Ninety-five per cent. of the binder aggregate shall pass a screen having circular openings the diameter of which shall be of three-quarters the thickness of the binder course to be laid. The remaining 5% shall not exceed in their smallest dimension the thickness of the binder course to be laid. The binder aggregate shall be graded from coarse to fine, so as to have the following mesh composition:

Passing 10-mesh sieve..........	15 to 35%	Total passing ½-in. screen, 35 to 85%.
Passing ½-in. screen and retained on 10-mesh sieve..........	20 to 50%	

The sand for the wearing course shall be carefully graded. For pavements to be subjected to medium or heavy traffic, there should be a preponderance of the finer particles; and for pavements to be subjected to light traffic, there may be a preponderance of the coarser particles. Specifications for a sand for wearing courses to be subjected to medium or heavy traffic should be similar to the following: The sand shall be hard, clean, and moderately sharp. On sifting it shall have the following mesh composition:

Passing 200-mesh	0 to 5%		
" 100-mesh and retained on 200-mesh	10 " 25%		Total passing 80-mesh and retained on 200-mesh, 20 to 40%.
" 80 " " " " 100 "	6 " 20%		
" 50 " " " " 80 "	5 " 40%		
" 40 " " " " 50 "	5 " 30%		
" 30 " " " " 40 "	5 " 25%		Total passing 10-mesh and retained on 40-mesh, 12 to 45%.
" 20 " " " " 30 "	5 " 15%		
" 10 " " " " 20 "	2 " 15%		

The filler should be thoroughly dry limestone dust, or dust from other equally satisfactory stone, or Portland cement, the whole of which should pass a 30-mesh sieve and at least 66% of which should pass a 200-mesh sieve. The surface mixture should contain from 6 to 20% of this filler, depending on the kind of sand and asphalt used and the traffic conditions on the street or streets to be paved.

The specifications should contain detailed requirements covering the physical properties of the asphalt cement, and should prescribe the bitumen content of the binder course and sheet-asphalt wearing course mixture. For the grading mentioned, the bitumen should be, for the close binder, from 4 to 7%; and for the sheet-asphalt wearing course mixture, from 9.5 to 13.5 per cent.

Construction.—The broken stone for the binder should be heated to a temperature between 107° cent. (225° Fahr.) and 177° cent. (350° Fahr.). The sand when mixed with the asphalt cement should have a temperature between 135° cent. (275° Fahr.) and 190° cent. (375° Fahr.). The asphalt cement when used should have a temperature between 121° cent. (250° Fahr.) and 177° cent. (350° Fahr.).

The asphalt cement and broken stone, or broken stone and sand, for the binder course, and the asphalt cement, sand, and filler for the wearing course, should be thoroughly mixed by machinery until a uniform mixture is produced in which all the particles are thoroughly coated with asphalt cement.

When brought to the work, the temperature of the binder mixture should be between 93° cent. (200° Fahr.) and 163° cent. (325° Fahr.), and of the wearing course mixture between 110° cent. (230° Fahr.) and 177° cent. (350° Fahr.). The binder course and the wearing surface should be compacted separately by rolling with a self-propelled roller weighing not less than 200 lb. per inch of width of tread, the rolling being carried on continuously at the rate of not more than 200 sq. yd. per hour per roller until a satisfactory compression is obtained. Excessive use of water on the steam roller while compacting the courses of the pavement should not be permitted. During the rolling of the wearing course, a small quantity of Portland cement should be swept over its surface. In cases where sheet-asphalt is constructed next to the curb, it is advisable to coat the surface for a space of 12 in. next to the curb with hot asphalt cement.

Stone Block Pavements.*

Materials.—The stone pavements of this country are generally of granite or sandstone, the particular kind being determined by the availability of the different materials. Limestone is used to a certain extent in one or two cities, but so slightly that it need not be considered.

In order to make a good paving block, stone should be resistant to wear, hard and tough, and of such a character as to be easily broken into regular shapes. Toughness is more important than hardness, as in very few cases will the blocks of a stone pavement, of a character such as is generally used, be much reduced under actual traffic. The quantity of wear is not as important as that the wear shall be uniform, so that the surface of the pavement may be kept smooth and even.

* For discussion of rates of grade, crown, artificial foundation, joints, etc., see "General Conclusions."

The character of sandstone is such that, though it does wear smooth, it is never slippery; but with granite, if the stone is too hard, even when no particular wear is noticed under traffic, the surface soon becomes smooth and slippery, and the harder the stone, the more slippery it becomes.

To make suitable paving blocks, granite should be a medium and uniform-grained stone, of such a character that when broken it will present smooth and even surfaces, and have a percentage of wear of not more than 4.5 and toughness of not less than 8. It should have a crushing strength of not less than 20 000 lb. per sq. in.

Sandstone should be hard and tough, and of a character to meet the requirements given for granite, except as to crushing strength, which should be not less than 16 000 lb. per sq. in.

As the stone must be made into comparatively small blocks, and as this is done by expensive labor, the size of the blocks is extremely important. Probably the ideal sized block would be 8 in. long, $3\frac{1}{2}$ in. wide, and, under general conditions, 5 in. deep, but if blocks were made to conform exactly to these dimensions, they would be extremely expensive, so that it is considered good practice to allow variations in length from 8 to 12 in., in width from $3\frac{1}{2}$ to $4\frac{1}{2}$ in., and in depth from $4\frac{1}{2}$ to $5\frac{1}{2}$ in., and on light-traffic streets, where certain conditions make a stone pavement desirable, an even less depth may be permitted. When sandstone is used, the blocks may be a little wider, as far as use is concerned. All blocks, however, should be sorted, so that the adjacent courses can be kept as nearly uniform in width as possible. Under no circumstances, however, should blocks of different widths be used in the same course. The blocks should be dressed so that they will lie with close joints, and have good, smooth, and even heads.

Another form of stone block pavement, used to some extent in Europe, and which has recently been introduced into this country, is known as "Durax" in England and as "Kleinpflaster" in Germany. It consists of blocks approximating cubes $2\frac{1}{2}$ to 4 in. in size, although they should not be exactly cubical; they should be sufficiently irregular, both in size and shape, to permit them to be laid in arcs of circles of comparatively small radii and so that the joints will not be excessively large. By laying the courses in circular arcs, none of the joints is parallel to any line of traffic.

In Europe these blocks are used to a great extent in resurfacing the broken stone roads where the traffic is too heavy for macadam, and to some extent in city streets. If the blocks can be produced in this country at a reasonable price, they will make very satisfactory roads. They have been used to a slight extent in pavements in some Southern cities.

During the last few years many pavements have been laid with granite blocks made by splitting up old ones which had been in use

for some years. With blocks that ranged from 4 to 5 in. in width, 10 to 12 in., and even 14 in., in length, and 8 in. in depth, it has been found possible to get many good blocks of smaller size by cutting them up. The new blocks, being small, could be cut to a reasonably true surface without much work, with the result that the old blocks recut would actually lay more square yards in a pavement than the original ones. This practice is to be commended, both on the score of economy and result.

Foundation and Cushion.—It is assumed that the foundation for permanent stone pavements will in all cases be concrete. On the concrete must be spread a material to act both as a cushion to the blocks themselves and to even up the surface of the concrete; and the smoother the surface and the less the variation in the depth of the blocks, the thinner can be the cushion, although it should not be less than ¾ in., in any event.

The cushion which has generally been used for this purpose is sand, but recently engineers have been considering the advisability of using Portland cement mortar instead. The objection to mortar, made by some, is that it makes a too solid base for the blocks, not giving any resiliency. This at present is a mooted question, and the Committee does not desire to express a positive opinion as to the relative values of the two. If a good bituminous cushion could be provided, it would probably be more satisfactory than either the mortar or the sand, but it is questionable if the advantage gained would justify the increase in expense. (See also Brick Pavements.)

Construction.—The blocks should be laid on the cushion stone to stone, keeping the joints as small as possible. The joints should be filled with water-proof material. For this purpose a Portland cement grout, asphalt, or some other bituminous filler is generally used, and in some cases sand is mixed with a bituminous material in order to increase the toughness of the filler. All these fillers give good results, but with cement grout the cost of taking up and restoring the pavement over cuts is increased over that of a bituminous filler, as in many cases blocks are broken in taking them up, and it is difficult to clean the cement from the individual blocks and also to keep the traffic from the cut or patch while the grout is setting after the pavement has been restored. Another disadvantage of the cement grout filler is that it is highly important that it be perfectly set before traffic is allowed on the pavement, and in large cities it is almost impossible to keep traffic from the pavement, after it is laid, for the necessary length of time.

Wood Block Pavements.*

General.—In using wood in pavements, special attention should be given to the crown of the street, as this material undoubtedly presents,

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

under certain conditions, a more slippery surface to traffic than any other. Wherever the longitudinal grade is sufficient to allow the water to run off freely, the crown should be very flat, not exceeding 3 in. in a roadway width of 30 ft. On streets which must be used continuously, the maximum grade allowed should not exceed 2%, although on residence streets, where pavements can be avoided when they are exceptionally slippery, grades up to 3 or 4% are permissible. In the Middle West, where, as a rule, there is less moisture in the air under ordinary conditions, the foregoing grades have been exceeded with satisfactory results.

Kinds of Wood for Blocks.—Whatever the kind of wood used for pavements, it must be treated with some preservative, in order to make it suitable. It is important that as many kinds of wood be utilized as possible, so that in any territory the most available one can be used. Just how many varieties can be utilized is uncertain at the present time, but those that are undoubtedly good are: Southern yellow pine, Douglas fir, tamarack, Norway pine, hemlock, and black gum. In the East and Central West, Southern yellow pine, and on the Pacific Coast, Douglas fir, are generally used. Experiments will be necessary to determine just what other kinds will be satisfactory.

The blocks must be sound, and must be well manufactured, square-butted, square-edged, free from unsound, loose or hollow knots, knot holes, worm holes, and other defects, such as shakes, checks, etc., that would be detrimental to the blocks.

The number of annual rings in the 1 in. which begins 2 in. from the pith of the block should not be less than 6, measured radially, provided, however, that blocks containing between 5 and 6 rings in this inch may be accepted if they contain 33⅓%, or more, of summer wood. In case the block does not contain the pith, the 1 in. to be used shall begin 1 in. away from the ring which is nearest to the heart of the block. The blocks in each charge shall contain an average of at least 70% of heart wood. No one block shall be accepted that contains less than 50% of heart wood.

Size of Blocks.—The blocks should be from 5 to 10 in. long, but should preferably average two times the depth. The Committee recommends blocks 4 in. in depth for very heavy traffic streets; blocks 3½ in. in depth for moderate traffic streets. For light traffic streets, 3 in. in depth may be used, but where 3-in. blocks are used, no blocks should be longer than 8 in. They may be from 3 to 4 in. in width, but, in any one city block, all of them should be of uniform width. A variation of $\frac{1}{16}$ in. should be allowed in the depth and ⅛ in. in width of the blocks from that specified. In all cases the width should be greater or less than the depth by at least ½ in.

Preservatives for Wood Blocks.—Many different materials have been used in the past for wood preservatives, but the Committee believes

that, taking all things into consideration, coal-tar creosote oil is the best. Some engineers, however, feel that a creosote oil produced from water-gas tar is as good as one produced from coal-gas tar, if not better. As the object of the preservative is not only to prevent the blocks from decay, but also to prevent them from swelling in wet weather or shrinking in dry weather, whatever the preservative, it should be of a character that will render the blocks stable and free from decay, for as long a time as possible. It is probable that under the traffic that prevails on most of the streets paved with wood in this country, if the blocks can be kept stable and free from decay, the pavement will last from 30 to 35 years, or even longer. It is necessary, however, to have an oil that is in itself stable and will remain in the blocks a long time.

It is thought that a heavy gravity oil will do this better than one of light gravity, as the former is less volatile and will maintain its condition better while exposed to atmospheric changes. The Committee recognizes that good results have been obtained by the use of a pure distillate oil, and also one which contains a certain quantity of coal-gas tar.

Treatment of Wood Blocks.—The timber may be either air-seasoned or green, but should preferably be treated within 3 months from the time it is sawed. Green timber and seasoned timber, however, should not be treated together in the same charge.

In any charge, blocks should contain at least 16 lb. of water-free oil per cubic foot of wood at the completion of the treatment. The blocks after treatment should show satisfactory penetration of the preservative, and in all cases the oil must be diffused throughout the sapwood. To determine this, at least 25 blocks shall be selected from various parts of each charge and sawed in half perpendicular to the fibers through the center, and if more than one of these blocks shows untreated sapwood, the charge should be re-treated. After re-treating, the charge should be again subjected to a similar inspection.

The surface of the blocks after treatment should be free from deposits of objectionable substances, and all blocks that have been materially warped, checked, or otherwise injured in the process of treatment, should be rejected.

Handling Blocks After Treatment.—Blocks should preferably be laid in the street as soon as possible after being treated. If they cannot be laid immediately, provision should be made to prevent them from drying out by stacking in close piles and covering them, and, if possible, by sprinkling them thoroughly at intervals. In any case, where they are not laid as soon as they are received on the street, they should be well sprinkled about 2 days before being laid, under the direction of the purchaser. It is important to have the wood sufficiently wet to be swelled to its maximum size before it is laid.

Inspection.—All material specified and processes used in the manufacture of the blocks therefrom should be subject to inspection, acceptance, or rejection at the plant of the manufacturer, which should be equipped with the necessary gauges, appliances, and facilities to enable the inspector to satisfy himself that the requirements of the specifications are fulfilled.

The purchaser should have the further right to inspect the blocks after delivery on the street, for the purpose of rejecting those that do not meet these specifications, except that the plant inspections should be final with respect to the kind of wood, rings per inch, oil, and treatment.

Construction.—There are two methods of laying wood blocks, one with and one without a cushion. In Europe it is invariably the practice to lay the blocks directly on the concrete bed. When this is done it is necessary that the surface of the concrete be made absolutely smooth and true to the required cross-section of the pavement. In this country the practice has been to surface-up the concrete, as has been mentioned in connection with stone blocks, with cement mortar or sand. The Committee believes that the cement mortar will give a better result than the sand. It should, however, be mixed as dry as possible and at the same time insure setting, and the blocks should be thoroughly rolled into it. If the sand cushion is used, the blocks should also be rolled to a smooth surface.

The Committee looks with a great deal of favor on the practice of finishing the concrete to a true surface and laying the blocks directly on it, and would suggest that engineers laying this pavement try this method, and, if the cost of producing a smooth concrete surface is not excessive, that the method be generally adopted.

The blocks should be laid closely and the joints filled with some suitable material. Three materials have been used in this country for joint filling: sand, cement grout, and a bituminous material. On heavy-traffic streets, if fine sand is used, good results will be obtained. On light-traffic streets, however, it may be better to use a bituminous filler of practically the same character as that used for granite. If a bituminous filler is used, the pavement should be covered with a thin layer of fine sand, which should be allowed to remain 1 or 2 weeks after the traffic has been allowed on the street. The Committee does not feel that cement grout should be used, in any case. Though a wood block pavement should keep stable, it is undoubtedly safer to use a bituminous joint along the curb to provide for expansion or contraction.

Bleeding.—Considerable inconvenience has been caused in certain cities by "bleeding", or the exudation of the preservative on the surface of the street, after the blocks have been laid. If the proper precaution is taken with the character of the material and the character

of the treatment, it is thought that this can be avoided. If the pavement, however, should bleed to such an extent as to be a nuisance, it should be covered with fine sand, so that the surface material can be absorbed. After one or two applications there should be no further trouble.

Definitions.

For the sake of uniformity and clearness, the Committee recommends that the following lists of terms of frequent use in expressions relating to highway work be recognized as having the meanings set forth in the list, unless otherwise definitely stated at any time by a user of such term or terms.

In the list given will be found some terms and definitions adopted by the American Society for Testing Materials noted thus *, others by the American Reporters on Communication No. 10 at the Third International Road Congress, designated thus †, and other terms and definitions which have been proposed by the Committee on "Standard Tests for Road Materials" (Committee D-4) of the American Society for Testing Materials, which have been indicated thus ‡. The Committee wishes to acknowledge here its obligations for suggestions thus reaching it.

Aggregate.—The inert material, such as sand, gravel, shell, slag, or broken stone, or combinations thereof, with which the cementing material is mixed to form a mortar or concrete.

Asphalt.—†Solid or semi-solid native bitumens, solid or semi-solid bitumens obtained by refining petroleums, or solid or semi-solid bitumens which are combinations of the bitumens mentioned with petroleums or derivatives thereof, which melt on the application of heat, and which consist of a mixture of hydrocarbons and their derivatives of complex structure, largely cyclic and bridge compounds.

Asphalt Block Pavement.—One having a wearing course of previously prepared blocks of asphaltic concrete.

Asphalt Cement.—A fluxed or unfluxed asphaltic material, especially prepared as to quality and consistency, suitable for direct use in the manufacture of asphaltic pavements, and having a penetration of between 5 and 250.

Asphaltenes.—† The components of the bitumen in petroleum, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulphide, but insoluble in paraffin naphthas.

Asphaltic.—Similar to, or essentially composed of, asphalt.

Base.—Artificial foundation.

Binder.—(1) A foreign or fine material introduced into the mineral portion of the wearing surface for the purpose of assisting the road metal to retain its integrity under stress, as well as, perhaps,

to aid in its first construction. (2) The course, in a sheet-asphalt pavement, frequently used between the concrete foundation and the sheet-asphalt mixture of graded sand and asphalt cement.

Bitumen.—* A mixture of native or pyrogenous hydrocarbons and their non-metallic derivatives, which may be gases, liquids, viscous liquids, or solids, and which are soluble in carbon disulphide.

Bituminous Cement.—A bituminous material suitable for use as a binder having cementing qualities which are dependent mainly on its bituminous character.

Bituminous Concrete Pavement.—One composed of broken stone, broken slag, gravel, or shell, with or without sand, Portland cement, fine inert material, or combinations thereof, and a bituminous cement incorporated together by a mixing method.

Bituminous Macadam Pavement.—One having a wearing course of macadam with the interstices filled by a penetration method with a bituminous binder.

Bituminous Material.—Material containing bitumen as an essential constituent.

> *Liquid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 50 grammes applied for 1 sec. of more than 350.
>
> *Semi-Solid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 100 grammes applied for 5 sec. of more than 10, and under a load of 50 grammes applied for 1 sec. of not more than 350.
>
> *Solid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 100 grammes applied for 5 sec. of not more than 10.

Bituminous Pavement.—One composed of broken stone, broken slag, gravel, shell, sand, or fine inert material, or combinations thereof, and bituminous cement incorporated together.

Bituminous Surface.—A superficial coat of bituminous material with or without the addition of stone or slag chips, gravel, sand, or material of similar character.

Blanket.—See "Carpet."

Bleeding.—The exudation of bituminous material on the roadway surface after construction.

Blown Petroleums.—† Semi-solid or solid products produced primarily by the action of air upon originally fluid native bitumens which are heated during the blowing process.

Bond.—The combined action of inertia, friction, and of the forces of adhesion and cohesion which helps the separate particles composing a crust or pavement to resist separation under stress. Mechanical bond is the bond produced almost wholly, in a well-built broken-stone

macadam road, by the interlocking of angular fragments of stone and the subsequent filling of the remaining interstices with the finer particles.

Bound.—Bonded.

Water-Bound.—Bonded with the aid of water.

Bituminous Bound.—Bonded with the aid of bituminous material.

Brick Pavement.—One having a wearing course of paving bricks or blocks.

Bridge.—A structure for the purpose of carrying traffic over a gap in the roadbed measuring 10 ft. or more in the clear span.

Camber of a Bridge.—The rise of its center above a straight line through its ends.

Camber of a Road.—See "Crown."

Carbenes.—† The components of the bitumen in petroleums, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulphide, but insoluble in carbon tetrachloride.

Carpet.—A bituminous surface of appreciable thickness, generally formed on top of a roadway by the application of one or more coats of bituminous material with gravel, sand, or stone chips added.

Cement.—An adhesive substance used for uniting particles of other materials to each other. Ordinarily applied only to calcined "cement rock", or to artificially prepared, calcined, and ground mixtures of limestone and silicious materials. Sometimes used to designate bituminous binder used in bituminous pavements, when the expression "bituminous cement" (q. v.) is understood to be meant.

Cement-Concrete.—An intimate mixture of gravel, shell, slag, or broken stone particles with certain proportions of sand or similar material, cement, and water, made previous to placing.

Cement-Concrete Pavement.—One having a wearing course of hydraulic cement concrete.

Cemented.—Bonded. Referring to water-bound macadam, the term "cemented" is used to designate that condition existing when, after rolling the stone forming the crust, the remaining voids have been filled with the finer sizes, and the stone dust or "flour" has, under the action of water, taken a "set", as does cement itself.

Chips.—Small angular fragments of stone or slag containing no dust.

Clay.—Finely divided earth, generally silicious and aluminous, which will pass a 200-mesh sieve. Also see "Gravel."

Coal-Tar.—† The mixture of hydrocarbon distillates, mostly unsaturated ring compounds, produced in the destructive distillation of coal.

Coat.—See "Carpet." (1) The total result of one or more single surface applications. (2) To apply a coat.

Coke-Oven Tar.—† Coal-tar produced in by-product coke ovens in the manufacture of coke from bituminous coal.

Consistency.—* The degree of solidity or fluidity of bituminous materials.

Course.—One or more layers of road metal spread and compacted separately for the formation of the road or pavement. Courses are usually referred to in the order of their laying as first course, second course, third course, etc. Also a single row of blocks in a pavement.

Crown.—The rise in cross-section from the lowest to the highest part of the finished roadway. It may be expressed either as so many inches (or tenths of a foot), or as a rate per foot of distance from side to center, *i. e.*, "the crown is 4 in.", or "the crown is ½ in. to the foot."

Crusher Run.—The total unscreened product of a stone crusher.

Crusher-Run Stone.—The product of a stone-crusher, unscreened except for the removal of the particles smaller than remaining on about a ¼-in. screen.

Crust.—That portion of a macadam or similar roadway above the foundation consisting of the road metal proper with its bonding agent or binder.

Culvert.—A structure for the purpose of carrying traffic over a gap in the road-bed, measuring less than 10 ft. in clear span.

Cut-Back Products.—Petroleum, or tar residua, which have been fluxed, each with its own or similar distillates.

Dead Oils.—* Oils, with a density greater than water, which are distilled from tars.

Dehydrated Tars.—† Tars from which all water has been removed.

Ditch.—The open-side drain of a roadway, usually deep in proportion to its width, and unpaved.

Drainage.—Provision for the disposition of water.

Side-Drainage.—That along the sides of the roadway.

Sub- or Under-Drainage.—That below the surface.

Surface Drainage.—That on the roadway or ground surface.

V-Drainage.—That provided by the construction of troughs in the sub-grade of the roadway which troughs are like a "V", with flat sloping sides, and are filled with stone.

Dust.—Earth or other matter in fine, dry particles, so attenuated that they can be raised and carried by air currents. The product of the crusher passing through a fine sieve.

Dust Layer.—Material applied to a roadway for temporarily preventing the formation or dispersion under traffic of distributable dust.

Earth Road.—A roadway composed of natural earthy material.

Emulsion.—A combination of water and oily material made miscible with water through the action of a saponifying or other agent.

Expansion Joint.—A separation of the mass of a structure, usually in the form of a joint filled with elastic material, which will provide opportunity for slight movement in the structure.

Fat.—Containing an excess. A fat asphalt mixture is one in which the asphalt cement is in excess and the excess is clearly apparent.

Filler.—(1) Relatively fine material used to fill the voids in the aggregate. (2) Material used to fill the joints in a brick or block pavement.

Fixed Carbon.—* The organic matter of the residual coke obtained upon burning hydrocarbon products in a covered vessel in the absence of free oxygen.

Flour.—† Finely ground rocks or minerals pulverized to an impalpable product.

Flush Coat.—See "Seal Coat."

Flushing.—(1) Completely filling the voids. (2) Washing a pavement with an excess of water.

Flux.—† Bitumens, generally liquid, used in combination with harder bitumens for the purpose of softening the latter.

Footway.—The portion of the highway devoted especially to pedestrians. A sidewalk.

Foundation.—The portion of the roadway below and supporting the crust or pavement.

Artificial Foundation.—That layer of the foundation especially placed on the sub-grade for the purpose of reinforcing the supporting power of the latter itself, and composed of material different from that of the sub-grade proper.

Free Carbon.—* In tars, organic matter which is insoluble in carbon disulphide.

Gas-House Coal-Tar.—† Coal-tar produced in gas-house retorts in the manufacture of illuminating gas from bituminous coal.

Grade.—(1) The profile of the center of the roadway, or its rate of rise or fall. (2) Elevation. (3) To establish a profile by cuts and fills or earthwork. (4) To arrange by sizes, broken stone, gravel, sand, or combinations of such materials.

Gravel.—Small stones or pebbles usually found in natural deposits more or less intermixed with sand, clay, etc., but in which mixture the particles which will not pass a 10-mesh sieve predominate.

Pea Gravel.—Clean gravel the particles of which approximate peas in size.

Grit.—Stone chips, slag chips, or small gravel.

Gutter.—The artificially surfaced and generally shallow waterway provided usually at the sides of the roadway for carrying surface drainage. Occasionally used synonymously with "ditch", but incorrectly so, as "gutters" are always paved or otherwise surfaced, and ditches are not.

Haunches.—The sides or flanks of a roadway. Sometimes also called "quarters."

Highway.—The entire right of way devoted to public travel, including the sidewalks and other public spaces, if such exist.

Layer.—A course made in one application.

Loam.—Finely divided earthy material containing a considerable proportion of organic matter.

Macadam.—A road crust composed of stone or similar material broken into irregular angular fragments compacted together so as to be interlocked and mechanically bound to the utmost possible extent.

Mastic.—A mixture of bituminous material and fine mineral matter suitably made for use in highway construction and for application in a heated condition.

Mat.—† See "Carpet."

Matrix.—* The binding material or mixture of binding material and fine aggregate in which the large aggregate is embedded or held in place.

Mesh.—The square opening of a sieve.

Metal.—See "Road-Metal."

Mortar.—A mixture of fine material such as sand, cement, and water or other liquid suitably proportioned and incorporated together for the purpose for which it is used.

Normal Temperature.—‡ As applied to laboratory observations of the physical characteristics of bituminous materials, is 25° cent. (77° Fahr.).

Oil-Gas Tars.—* Tars produced by cracking oil vapors at high temperatures in the manufacture of oil gas.

Palliative.—A short-lived dust layer.

Patching.—Repairing or restoring small isolated areas in the surface of the metaled or paved portion of the highway.

Pavement.—The wearing course of the roadway or footway, when constructed with a cement or bituminous binder, or composed of blocks or slabs, together with any cushion or "binder" course.

Penetration.—In laboratory investigations, the distance, expressed in hundredths of a centimeter, entered a sample by a No. 2 cambric needle, operated in a machine for the purpose, and under known conditions of loading, time, and temperature. Where the conditions of test are not specifically mentioned, the load, time, and temperature are understood to be 100 grammes, 5 sec., and 25° cent. (77° Fahr.), re-

spectively, and the units of penetration to indicate hundredths of a centimeter.

Penetration Method.—The method of constructing a bituminous-macadam pavement by pouring or grouting the bituminous material into the upper course of the road metal before the binding of the latter has been completed.

Pitch.—† Solid residue produced in the evaporation or distillation of bitumens, the term being usually applied to residue obtained from tar.

Hard Pitch.—Pitch showing a penetration of not more than ten.

Soft Pitch.—Pitch showing a penetration of more than ten.

Straight-Run Pitch.—‡ A pitch run in the initial process of distillation, to the consistency desired without subsequent fluxing.

Pocket.—A hole or depression in the wearing course.

Pot-Hole.—A hole extending below the wearing course.

Profile.—A longitudinal section of a highway, generally taken along the center line.

Quarters.—The four sections of equal width which, side by side, make up the total width of a roadway.

Raveling.—The loosening of the metal composing the crust.

Refined Tar.—† A tar freed from water by evaporation or distillation which is continued until the residue is of desired consistency or a product produced by fluxing tar residuum with tar distillate.

Renewals.—Extensive repairs over practically the whole surface of the metaled or paved portion of the highway.

Repairs.—The restoration or mending of a considerable amount of the metaled or paved portion of the highway, but not usually of a majority of the surface area. More extensive than "Patching" but less so than "Renewals."

Resurfacing.—The renewal of the surface of the crust or pavement.

Road.—A highway outside of an urban district.

Road-Bed.—The natural foundation of a roadway.

Road Metal.—Broken stone, gravel, slag, or similar material used in road and pavement construction and maintenance.

Roadway.—That portion of a highway particularly devoted to the use of vehicles.

Rock Asphalt.—Sandstone or limestone naturally impregnated with asphalt.

Rock Asphalt Pavement.—A wearing course composed of broken or pulverized rock asphalt with or without the addition of other bituminous materials.

Sand.—Finely divided rock detritus the particles of which will pass a 10-mesh and be retained on a 200-mesh screen. Also see "Gravel."

Sand-Clay Road.—A roadway composed of an intimate mixture of sand and clay.

Scarify.—To loosen and disturb superficially.

Screen.—In laboratory work an apparatus, in which the apertures are circular, for separating sizes of material.

Screenings.—Broken rock of a size that will pass through a ½- to ¾-in. screen, depending on the character of the stone.

Seal Coat.—A final superficial application of bituminous material during construction to a bituminous pavement.

Setting Up.—As applied to bituminous material, the relative quick change which takes place after its application to a roadway, indicated by its hardening after cooling and exposure to atmospheric and traffic conditions, as opposed to the slower changes later occurring gradually and almost imperceptibly.

Shaping.—Trimming up and preparing a sub-grade preparatory to applying the first course of the road metal or artificial foundation.

Sheet-Asphalt Pavement.—One having a wearing course composed of asphalt cement and sand of predetermined grading, with or without the addition of fine material, incorporated together by mixing methods.

Sheet Pavement.—A pavement free from frequent joints such as would accompany small slabs or blocks, and which has an appreciable thickness (say, in excess of 1 in. on the average) for its wearing course.

Shoulders.—The portion of the highway between the edges of the road metal or pavement and the gutters, slopes, or watercourses.

Side Drain.—See "Drainage."

Sidewalk.—The portion of the highway reserved for pedestrians.

Sieve.—In laboratory work an apparatus, in which the apertures are square, for separating sizes of material.

Silt.—Naturally deposited fine earthy material, which will pass a 200-mesh sieve.

Slag.—‡ Fused or partly fused compounds of silica in combination with lime or other bases, resulting in secondary products from the reduction of metallic ores.

Spalls.—Fragments broken off by a blow, irregular in shape, and of sufficient size to be comparable to the original mass.

Squegee.—A tool with a rubber or leather edge for scraping or cleaning hard surfaces, or for spreading and distributing liquid material over and into the superficial interstices of roadways.

Squegee Coat.—An application by means of the squegee.

Stone Block Pavement.—One having a wearing course composed of stone blocks quite or nearly rectangular in shape.

Street.—A highway in an urban district.

Sub-Grade.—The upper surface of the native foundation on which is placed the road metal or the artificial foundation, in case the latter is provided.

Superficial Coat.—A light surface coat.

Surface Coat.—See "Carpet."

Surface Treatment.—Treating the finished surface of a roadway with bituminous material.

Surfacing.—(1) The crust or pavement. (2) Constructing a crust or pavement. (3) Finally finishing the surface of a roadway. (4) Treating the surface of a finished roadway with a bituminous material.

Tailings.—Stones which after going through the crusher do not pass through the largest openings of the screens.

Tar.—† Bitumen which yields pitch upon fractional distillation and which is produced as a distillate by the destructive distillation of bitumens, pyro-bitumens, or organic material.

Telford.—Properly an artificial foundation advocated by Thomas Telford (1757-1820), and consisting of a pavement of stone about 8 in. thick, laid by hand, and closely packed and wedged together. The individual stones were desired to be about 16 sq. in. in section, and about 8 in. in length. They were set close together on the prepared sub-grade, their longest dimension vertical and on their larger ends, their interstices chinked with smaller stones, and the whole rammed (or rolled) until firm and unyielding.

Telford Macadam.—Macadam with an artificial foundation of Telford.

Under-Drain.—See "Drainage."

Up-Keep.—Maintenance.

V-Drain.—See "Drainage."

Viscosity.—‡ The measure of the resistance to flow of a bituminous material, usually stated as the time of flow of a given quantity of the material through a given orifice.

Volatile.—Applied to those fractions of bituminous materials which will evaporate at climatic temperatures.

Water-Bound.—Bound or bonded with the aid of water.

Water-Gas Tars.—* Tars produced by cracking oil vapors at high temperatures in the manufacture of carburetted water-gas.

Wearing Coat.—The superficial layer of the crust or pavement exposed to traffic.

Wearing Course.—The course of the crust or pavement exposed to traffic..

Wood Block Pavement.—One having a wearing course composed of wood paving blocks, generally rectangular in shape.

The Committee wishes to express again its deep appreciation of the assistance rendered it by the Board of Direction and by your Secretary, Dr. Chas. Warren Hunt, as well as by members of the Society, and others.

Very respectfully,

For the Special Committee on Materials for Road Construction and on Standards for Their Test and Use.

ARTHUR H. BLANCHARD,
Secretary.

COMMITTEE:

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OCTOBER 27TH, 1917.

APPENDIX A

FORM OF RECORD FOR DATA CONCERNING THE USE OF HIGHWAY MATERIALS

(The following forms of records are recommended for the use of Highway Engineers. They cover both bituminous and non-bituminous materials. Although combined in a single table, it is believed that the data for different kinds of road surfaces can advantageously be placed on separate sheets or cards for convenient filing and reference.)

GENERAL INFORMATION.

State............County..............Town or City..............
Road or street...............Limits of improvement..............
Length of improvement, in feet..............................
Width of crust or pavement, in feet (average)..................
Area of crust or pavement, in square yards......................
Kind of surface and foundation................................
Percentage of grade, maximum—per cent....Minimum—per cent....
Amount of crown, maximum..., minimum..., Nature of sub-grade...
Maximum and minimum air temperature during year...............
Hours of working day............Labor wage per hour............
Contractor ....
Dates of beginning and completion of improvement................
Class of highway or nature of traffic..........................

TRAFFIC CENSUS FOR........ HOURS, BEING THE AVERAGE OF .... OBSERVATIONS TAKEN BETWEEN THE HOURS OF........ AND........ ON...

LOCATION OF POINT OF OBSERVATION...............................

	Commercial Vehicles.			Passenger vehicles.
	Empty.	Loaded.	Estimate (in pounds) of maximum load per inch of tire.	
One-horse vehicles..................				
Two or three-horse vehicles..........				
Four or more horse vehicles..........				
Motor cycles..........................				
" runabouts				
" touring cars (open or closed).				
" busses				
" trucks............................				

Construction and Cost Details.

A.—*Foundation.*

1. Material ....
2. Thickness ....
3. Cost per square yard..
4. Estimated life, in years..

B.—*Wearing Course.*

1. Material ....
2. Thickness ....
3. Size of block or brick..
4. Kind and amount of bituminous cement..............................
5. Kind of joint..
6. Proportions of aggregate..
7. Cushion or binder course..
8. First cost per square yard..
9. Life, in years..
10. Average annual maintenance cost per square yard during life of wearing course ....

C.—*Traffic Data.*

1. Tons per year (2 000 lb.)..
2. Average tons per yard of width......................................
3. Proportion of tonnage on metal tires................................
4. " " *C*-3 on tires 2 in. or less in width..........................

Items of Cost for Each Square Yard.

	Foundation	Wearing Course
Materials..............................		
Labor..................................		
Superintendence......................		
Overhead, including interest on plant, depreciation, etc..............		
....		

Data Per Square Yard.

A-3.—First cost of foundation..
A-5.—Annual interest and sinking fund for foundation.....................
A-6.—Total annual cost of foundation......................................
B-10.—Annual maintenance cost of wearing course..........................
B-11.—Annual interest and sinking fund for wearing course................
B-12.—Total annual cost of wearing course..................................

Yearly cost per 1 000 tons of traffic $= \dfrac{A\text{-}6 + B\text{-}12}{C\text{-}2}\ 1\,000 =$ ....

(The following data or such parts as apply to a particular road or street could be incorporated on the sheet or card containing the data

immediately preceding, or could be placed on a separate sheet or card containing the results of tests and analyses.)

Broken Stone and Broken Slag.

Name and origin....................................
Specific gravity....................................
Absorption of water per cubic foot....................................
Abrasion, percentage of loss....................................
Toughness....................................
Cementation....................................
Crushing strength per square inch....................................
Mechanical analysis.................. (Use table under this title)
Voids, percentage of, loose and compacted....................................

Gravel.

Location....................................
Specific gravity....................................
Abrasion, percentage of loss....................................
Cementation....................................
Mechanical analysis.................. (Use table under this title)
Voids, percentage of, loose and compacted....................................

Sand.

Location....................................
Specific gravity....................................
Mechanical analysis.................. (Use table under this title)
Voids, percentage of, loose and compacted....................................
Tensile strength in cement briquettes, as compared with standard Ottawa sand....................................

Mixtures of Sand or Other Fine Highway Materials with Broken Stone, Broken Slag, or Gravel.

Specific gravity....................................
Mechanical analysis.................. (Use table under this title)
Voids, percentage of, loose and compacted....................................

Paving Brick.

Composition....................................
Name of manufacturer....................................
Rattler test, percentage of loss....................................

Stone Block.

Name and origin....................................
Specific gravity....................................
Absorption of water per cubic foot....................................
Abrasion, percentage of loss....................................
Toughness....................................
Hardness....................................
Crushing strength per square inch....................................

Wood Block.

Character of wood..
Weight of blocks..
Soundness..
Rings per radial inch..
Quantity of preservative..
Absorption of water after treatment..
Character of preservative:
  Specific gravity at 25° cent. (77° Fahr.)..
  Specific gravity at 38° cent. (100° Fahr.)..
  Solubility in benzol or chloroform..
  Water content..
Distillation:
    Up to 170° cent..
    170° to 200° cent..
    200° to 210° cent..
    210° to 235° cent..
    235° to 270° cent..
    270° to 300° cent..
    300° to 315° cent..
    315° to 355° cent..

MECHANICAL ANALYSIS.

	Percentages by weight.	
Passing 200-mesh sieve........		
“ 100 “ “ retained on 200-mesh sieve		
“ 80 “ “ “ “ 100 “ “		
“ 50 “ “ “ “ 80 “ “		
“ 40 “ “ “ “ 50 “ “		
“ 30 “ “ “ “ 40 “ “		
“ 20 “ “ “ “ 30 “ “		
“ 10 “ “ “ “ 20 “ “		
“ ¼-in. screen, “ “ 10 “ “		
“ ½ “ “ “ “ ¼-in. screen		
“ ¾ “ “ “ “ ½ “ “		
“ 1 “ “ “ “ ¾ “ “		
“ 1¼ “ “ “ “ 1 “ “		
“ 1½ “ “ “ “ 1¼ “ “		
“ 2 “ “ “ “ 1½ “ “		
“ 2½ “ “ “ “ 2 “ “		
“ 3 “ “ “ “ 2½ “ “		
“ 3½ “ “ “ “ 3 “ “		

Portland Cement.

Loss on ignition, percentage........ ....
Insoluble residue, percentage........ ....
Specific gravity........ ....
Retained on 200-mesh sieve, percentage.. ....
Retained on 100-mesh sieve, percentage.. ....
Steam test........ ....
Initial set, time, in minutes........ ....
Final set, time, in minutes........ ....
Tensile strength, neat, 24 hours........ ....
Tensile strength, neat, 7 days........ ....
Tensile strength, 1: 3 Ottawa sand, 7 days. ....
Tensile strength, 1: 3 Ottawa sand, 28 days ....
Compressive strength, per square inch..... ....
Constancy of volume........ ....

BITUMINOUS MATERIALS.

The following forms are given as illustrations of those to be used for recording the properties of bituminous materials.

Asphalt Cements for Bituminous Macadam, Bituminous Concrete, Asphalt Block and Sheet-Asphalt Pavements and Fillers for Brick and Stone Block Pavements.

Trade name........
Manufacturer........
General characteristics........
Specific gravity at 25° cent. (77° Fahr.)........ ....
Flash point........ ....
Solubility in CS_2 (carbon disulphide)........ ....
  Organic matter insoluble........ ....
  Inorganic matter insoluble........ ....
Solubility of bitumen in CCl_4 (carbon tetrachloride)...... ....
Solubility of bitumen in petroleum naphtha........ ....
Penetration 4° cent. (39° Fahr.), 200 grammes, 1 min..... ....
Penetration 25° cent. (77° Fahr.), 100 grammes, 5 sec..... ....
Penetration 46° cent. (115° Fahr.), 50 grammes, 5 sec..... ....
Float test........ ....
Melting point by ring and ball method........ ....
Ductility at 4° cent. (39° Fahr.)........ ....
Ductility at 25° cent. (77° Fahr.)........ ....
Fixed carbon content........ ....
Paraffin content........ ....
Loss on evaporation at 163° cent. (325° Fahr.), 5 hours.... ....
  Penetration of residue 4° cent. (39° Fahr.), 200 grammes, 1 min...
  Penetration of residue 25° cent. (77° Fahr.), 100 grammes, 5 sec....

Penetration of residue 46° cent. (115° Fahr.), 50 grammes, 5 sec.....
Melting point of residue, by ring and ball method...............
Float test on residue.......................................
Ductility of residue at 4° cent. (39° Fahr.)
Ductility of residue at 25° cent. (77° Fahr.)

Tar Cements for Bituminous Macadam and Bituminous Concrete Pavements and Fillers for Brick and Stone Block Pavements.

Trade name..
Manufacturer..
General characteristics.....................................
Water...
Specific gravity at 25° cent. (77° Fahr.)...................
Flash point...
Solubility in CS_2 (carbon disulphide).......................
Specific viscosity, Engler..................................
Melting point, by cube method...............................
Float test..
Distillation by weight and by volume........................
Up to 110° cent...
110° to 170° cent...
170° to 235° cent...
235° to 270° cent...
270° to 300° cent...
Specific gravity of total distillate at 25° cent. (77° Fahr.)....
Melting point of residue, by cube method....................
Float test on residue.......................................

APPENDIX B

TESTS OF NON-BITUMINOUS MATERIALS.

It is recommended that the following methods for performing tests of non-bituminous materials be adopted as standards:

SPECIFIC GRAVITY OF COARSE AGGREGATES.*

The apparent specific gravity shall be determined in the following manner:

The sample, weighing 1 000 grammes and composed of pieces approximately cubical or spherical in shape and retained on a screen having 1.27-cm. (½-in.) circular openings, shall be dried to constant weight at a temperature between 100 and 110° cent. (212 and 230° Fahr.), cooled, and weighed to the nearest 0.5 gramme. Record this weight as weight *A*. In the case of homogeneous material, the smallest particles in the sample may be retained on a screen having 1¼-in. circular openings.

Immerse the sample in water for 24 hours, surface-dry individual pieces with aid of a towel or blotting paper, and weigh. Record this weight as weight *B*.

Place the sample in a wire basket of approximately ¼-in. mesh, and about 12.7 cm. (5 in.) square and 10.3 cm. (4 in.) deep, suspend in water† from center of scale pan, and weigh. Record the difference between this weight and the weight of the empty basket suspended in water as weight *C*. (Weight of saturated sample immersed in water.)

The apparent specific gravity shall be calculated by dividing the weight of the dry sample (*A*) by the difference between the weights of the saturated sample in air (*B*) and in water (*C*), as follows:

$$\text{Apparent specific gravity} = \frac{A}{B - C}.$$

Attention is called to the distinction between apparent specific gravity and true specific gravity. Apparent specific gravity includes the voids in the specimen, and is, therefore, always less than or equal to, but never greater than, the true specific gravity of the material.

APPARENT SPECIFIC GRAVITY OF SAND, STONE SCREENINGS, OR OTHER FINE HIGHWAY MATERIAL.

Apparatus.—The determination shall be made with a Jackson specific gravity apparatus which shall consist of a burette, with graduations reading to 0.01 in specific gravity, about 23 cm. (9 in.) long and

* Proposed in 1917 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

† The basket may be conveniently suspended by a fine wire hung from a hook shaped in the form of a question mark with the top end resting on the center of the scale pan.

with an inside diameter of about 0.6 cm. (0.25 in.), which shall be connected with a glass bulb approximately 13 cm. (5.5 in.) long and 4.5 cm. (1.75 in.) in diameter, the glass bulb being of such size that from a mark on the neck at the top to a mark on the burette just below the bulb, the capacity is exactly 180 c.c. (6.09 oz.); and an Erlenmeyer flask which shall contain a hollow ground-glass stopper having the neck of the same bore as the burette and a capacity of exactly 200 c.c. (6.76 oz.) up to the graduation on the neck of the stopper.

Method of Determination.—The method shall consist of: First, dry at not more than 110° cent. (230° Fahr.) to a constant weight a sample weighing about 55 grammes; second, weigh to 0.1 gramme, 50 grammes of the dry sample and pour it into the unstoppered Erlenmeyer flask; third, fill the bulb and burette with kerosene, leaving just space enough to take the temperature by introducing a thermometer through the neck; fourth, remove the thermometer and add sufficient kerosene to fill exactly to the mark on the neck, drawing off any excess with the burette; fifth, run into the flask about one-half of the kerosene in the bulb to remove air bubbles and then run in more kerosene, removing any material adhering to the neck of the flask, until the kerosene is just below the ground glass; sixth, place the hollow ground-glass stopper in position and turn it to fit tightly, and then run in kerosene exactly to the 200-c.c. (6.76-oz.) graduation on the neck, care being taken to remove all air bubbles in the flask; seventh, read the specific gravity from the graduation on the burette, and the temperature of the oil in the flask, noting the difference between the temperature of the oil in the bulb before the determination and that of the oil in the flask after the determination; eighth, make a temperature correction to the reading of the specific gravity in accordance with the table furnished by the manufacturer of the apparatus, adding the correction if the temperature of the kerosene has increased and subtracting it if the temperature of the kerosene has decreased.

Absorption of Water Per Cubic Foot of Rock.*

"The absorption of water per cubic foot of rock shall be determined by the following method: First, a sample weighing between 29 and 31 g. and approximately cubical in shape shall be dried in a closed oven for 1 hour at a temperature of 110° C. (230° F.) and then cooled in a desiccator for 1 hour; second, the sample shall be rapidly weighed in air; third, trial weighings in air and in water of another sample of approximately the same size shall be made in order to determine the approximate loss in weight on immersion; fourth, after the balances shall have been set at the calculated weight, the first sample shall be weighed as quickly as possible in distilled water having a temperature of 25° C. (77° F.); fifth, allow the sample to remain 48 hours in distilled

* Proposed in 1914 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

water maintained as nearly as practicable at 25° C. (77° F.), at the termination of which time bring the water to exactly this temperature and weigh the sample while immersed in it; sixth, the number of pounds of water absorbed per cubic foot of the sample shall be calculated by the following formula:

$$\text{Pounds of water absorbed per cubic foot} = \frac{W_2 - W_1}{W - W_1} \times 62.24,$$

in which W = the weight in grammes of sample in air, W_1 = the weight in grammes of sample in water just after immersion, W_2 = the weight in grammes of sample in water after 48 hours immersion, and 62.24 = the weight in pounds of a cubic foot of distilled water having a temperature of 25° C. (77° F.).

"Finally, the absorption of water per cubic foot of the rock, in pounds, shall be the average of three determinations made on three different samples according to the method above described."

Abrasion Test for Broken Stone or Broken Slag.*

"The machine shall consist of one or more hollow iron cylinders; closed at one end and furnished with a tightly fitting iron cover at the other; the cylinders to be 20 cm. [7.87 in.] in diameter and 34 cm. [13.38 in.] in depth inside. These cylinders are to be mounted on a shaft at an angle of 30° with the axis of rotation of the shaft.

"At least [13.6 kg.] 30 lb. of coarsely broken stone shall be available for a test. The rock to be tested shall be broken in pieces as nearly uniform in size as possible, and as nearly 50 pieces as possible shall constitute a test sample. The total weight of rock in a test shall be within 10 grammes of 5 kilogrammes [11.02 lb.]. All test pieces shall be washed and thoroughly dried before weighing. 10 000 revolutions, at the rate of between 30 and 33 to the minute, must constitute a test. Only the percentage of material worn off which will pass through a 0.16 cm. (1-16 inch) mesh sieve shall be considered in determining the amount of wear. * * *"

Abrasion Test for Gravel.

Note.—As tests to determine the loss on abrasion of gravel are in an experimental stage, the Committee has included two methods which have been used and are being investigated. In noting the results obtained, the Committee advises stating the method used.

Method No. 1.—The test for abrasion of gravel shall be made with a Deval abrasion machine. (See "Abrasion Test for Broken Stone or Broken Slag.")

A charge of gravel shall consist of pieces which shall pass a screen having circular openings 5.08 cm. (2 in.) in diameter and be retained on a screen having circular openings 1.27 cm. (½ in.) in diameter. The total weight of gravel in a charge shall be within 10 grammes of 5 kg. (11.02 lb.). The gravel to compose a charge shall be washed,

* Method adopted by the Am. Soc. for Testing Materials, August 15th, 1908.

and dried in a closed oven for 1 hour at a temperature within 5° of 110° cent. (230° Fahr.). The charge of gravel shall be placed in one cylinder of the machine, which shall be rotated at a rate of not less than 30 nor more than 33 rev. per min. Ten thousand revolutions shall constitute a test. The percentage of material worn off which will pass through a sieve having openings of 0.16 cm. ($\frac{1}{16}$ in.) shall be considered the amount of wear of the charge of gravel. The loss by abrasion, determined as stated, shall be expressed in terms of the percentage of the total weight of the charge of gravel.

*Method No. 2.**—The aggregate is first screened through screens having circular openings 2 in., 1 in., and ½ in. in diameter. The sizes used for this test are divided equally between those passing the 2-in. and retained on the 1-in. screen, and those passing the 1-in. and retained on the ½-in. screen. The material of these sizes is washed and dried. The following weights of the dried stone are then taken: 2 500 grammes of the size passing the 2-in. and retained on the 1-in. screen, and 2 500 grammes of the size passing the 1-in. and retained on the ½-in. screen. This material is placed in the cast-iron cylinder of the Deval machine, as specified for the standard abrasion on stone. Briefly described, this machine consists of a frame and two or more cylinders mounted at an angle of 30° with the axis of rotation. The cylinders are 20 cm. in diameter and 34 cm. deep, inside dimensions. Six cast-iron spheres, 1.875 in. in diameter and weighing approximately 0.95 lb. (0.45 kg.) each, are placed in the cylinder as an abrasive charge. (The iron composing these spheres is the same as that used for the spheres in the Standard Paving Brick Rattler Test.) After the cast-iron spheres have been placed in the cylinder the lid is bolted on and the cylinder is mounted in the frame of the Deval machine. The duration of the test and the rate of rotation are the same as specified for the standard test for stone, namely, 10 000 revolutions at a rate of from 30 to 33 rev. per min. At the completion of the test the material is taken out and screened through a 16-mesh sieve. The material retained on the sieve is washed and dried and the percentage of loss by abrasion of the material passing the 16-mesh sieve is calculated. When the material has a specific gravity of less than 2.20, a total weight of 4 000 grammes, instead of 5 000 grammes, shall be used in the abrasion test.

Toughness Test for Rock or Slag.†

Definition.—Toughness, as applied to rock, is the resistance offered to fracture under impact, expressed as the final height of blow required of a standard hammer to cause fracture of a cylindrical test specimen of given dimensions.

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* Proposed by the First Conference of State Highway Testing Engineers and Chemists held at the U. S. Office of Public Roads in February, 1917.

† Proposed in 1917 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

Sampling.—Quarry samples of rock from which test specimens are to be prepared shall measure at least 6 in. on a side and at least 4 in. in thickness, and, when possible, shall have the plane of structural weakness* of the rock plainly marked thereon. Samples should be taken from freshly quarried material, and only from pieces which show no evidences of incipient fracture due to blasting or other causes. The samples should preferably be split from large pieces by the use of plugs and feathers, and not by sledging. Commercial stone-block samples from which test specimens are to be prepared shall measure at least 3 in. on each edge.

Size and Form of Test Specimen.—Specimens for test shall be cylinders prepared as described in the next paragraph, 25 mm. in height and from 24 to 25 mm. in diameter. Three test specimens shall constitute a test set. The ends of the specimens shall be plane surfaces at right angles to the axis of the cylinder.

Preparation of Test Specimens.—One set of specimens shall be drilled perpendicular and another parallel to the plane of structural weakness of the rock, if such plane is apparent. If a plane of structural weakness is not apparent, one set of specimens shall be drilled at random. Specimens shall be drilled in a manner which will not subject the material to undue stresses and will insure the specified dimensions.† The ends of the cylinders may be sawed with a band or diamond saw,‡ or in any other way which will not induce incipient fracture, but shall not be chipped or broken off with a hammer. After sawing, the ends of the specimens shall be ground plane with carborundum or emery on a cast-iron lap until the cylinders are 24 mm. in length.

Impact Machine.—Any form of impact machine which will comply with the following essentials may be used in making the test: A cast-iron anvil weighing not less than 50 kg., firmly fixed on a solid foundation; a hammer weighing 2 kg., arranged so as to fall freely between suitable guides; a plunger made of hardened steel, and weighing 1 kg., arranged to slide freely in a vertical direction in a sleeve, the lower end of the plunger being spherical in shape, with a radius of 1 cm.; means for raising the hammer and for dropping it on the plunger from any specified height from 1 to not less than 75 cm., and means for determining the height of fall to approximately 1 mm.; means for holding the cylindrical test specimen securely on the anvil without rigid lateral support, and under the plunger in such a way that the center of its upper surface, throughout the test, shall be tangent to the spherical end of the plunger at its lowest point.

* The plane of structural weakness, in certain cases, may be the rift, cleavage, or bedding plane.

† The form of diamond drill described in *Bulletin No. 347*, U. S. Department of Agriculture, pp. 6–7, is recommended, and should prove satisfactory, if the instructions are strictly followed.

‡ A satisfactory form of diamond saw is described in *Bulletin No. 347*, U. S. Department of Agriculture, pp. 7–9.

Method of Testing.—The test shall consist of a 1-cm. fall of the hammer for the first blow, a 2-cm. fall for the second blow, and an increase of 1-cm. fall for each succeeding blow, until failure of the test specimen occurs.

Recording and Reporting Results.—The height of the blow, in centimeters, at failure shall be the toughness of the test specimen. The individual and the average toughness of three test specimens shall be reported when no plane of structural weakness is apparent. In cases where a plane of structural weakness is apparent, the individual and average toughness of the three specimens in each set shall be reported and identified. Any peculiar condition of a test specimen which might affect the result, such as the presence of seams, fissures, etc., shall be noted and recorded with the test result.

Hardness Test for Rock or Slag.

The test for hardness shall be made with a "Dorry", or similar machine, consisting of a revolving disk on which is fed, at a uniform rate, a standard quartz sand passing a 30- and retained on a 40-mesh sieve. Two cores, each 25 mm. (0.98 in.) in diameter, shall be cut from the material to be tested, and their faces ground off so as to be at right angles to the long axes of the cores. The cores shall be placed in the holders or dies and weighted so that the entire weight of each core with its holder and added weight is 1 250 grammes. Each core shall be ground in the machine on one face for 1 000 revolutions, after which it shall be reversed and ground on the other face for an equal number of revolutions. The loss of weight of each specimen shall be determined at the end of each 1 000 revolutions, and the average loss in weight shall be used for stating the hardness of the material, which latter shall be expressed by the formula: Hardness $= 20 - \frac{1}{3} W$, where W equals the average loss, in grammes per 1 000 revolutions.

Cementation of Rock, Slag, and Gravel Powders.

The cementation test shall be made as follows: Of the material to be tested, 500 grammes shall be broken to pass a 1.27-cm. ($\frac{1}{2}$-in.) mesh sieve and then placed in a ball mill with 90 c.c. (3.04 oz.) of water and two steel shot weighing together 9 kg. (20 lb.). The mill and its charge shall be revolved for $2\frac{1}{2}$ hours at a rate of 2 000 rev. per hour. The dough thus formed shall then be removed, and 25 grammes of an average sample of it shall be placed in a metal die, 25 mm. (0.98 in.) in diameter, and subjected to a pressure of 132 kg. per sq. cm. for an instant in a hydraulic press. The cylindrical briquette resulting should measure exactly 25 mm. (0.98 in.) in height. If it does not, subsequent samples of the dough shall be taken in such quantity that the resulting briquette after compression will be exactly

25 mm. (0.98 in.) in height. Five such briquettes shall be made and allowed to dry in the air for a period of 20 hours, after which they shall be heated for 4 hours in a hot-air oven at a temperature of 93.3° cent. (200° Fahr.), and then cooled in a desiccator for 20 min. These cylinders or briquettes shall then be tested in a machine, as follows:

The machine shall be arranged so that a 1-kg. (2.20-lb.) hammer is raised to a height of 1 cm. (0.39 in.) and then falls freely on a plunger transmitting the shock of the blows of the hammer through the plunger to the test piece, successive blows being struck by the hammer at a rate of 40 to 70 per min., until the test piece fails, which is indicated by the failure of the plunger or hammer to rebound. The test piece shall be placed on the anvil under the plunger without lateral support, and may be fastened in place on the anvil by a drop of shellac. The average of the number of blows on the five briquettes, required to produce failure in each case, is the result to be reported, and is the "coefficient of cementation."

Crushing Strength of Rock or Slag.

Cylinders shall be cut from a suitable block of the material to be tested, each of which cylinders shall be, as nearly as practicable, 5 cm. (2 in.) in diameter and 10 cm. (4 in.) in length. After cutting, the dimensions of each cylinder shall be accurately measured and recorded. Each cylinder shall then be subjected to compression, and the ultimate stress at which its failure occurs shall be noted. This stress divided by the average area in cross-section of the cylinder in square inches shall be reported. It is desirable that the test of the material shall be made on at least three such cylinders separately, and the average of the three or more specimens shall be taken as the average resistance to crushing of the material. In making the test, the cylinder shall be fixed in the testing machine so as to be unsupported on its sides and rest squarely on its ends, and the compressive stress shall be applied cumulatively. The ends of the cylinder shall be at right angles to its long axis, and the blocks or pieces of the machine in contact with the ends of the cylinder and through which the pressure is transmitted shall have such position and freedom of movement in the machine as will insure the application of the stress directly along or parallel to the long axis of the cylinder.

Mechanical Analysis of Broken Stone, Broken Slag, or Gravel.*

The method shall consist of, first, drying at not more than 110° cent. (230° Fahr.), to a constant weight, a sample weighing in pounds six times the diameter in inches of the largest holes required; second, passing the sample through such of the following sized screens having circular openings as are required or called for by the specification, the screens to be used in the order named: 8.89 cm. (3½ in.), 7.62 cm.

* Method adopted in 1916 by the Am. Soc. for Testing Materials.

(3 in.), 6.35 cm. (2½ in.), 5.08 cm. (2 in.), 3.81 cm. (1½ in.), 3.18 cm. (1¼ in.), 2.54 cm. (1 in.), 1.90 cm. (¾ in.), 1.27 cm. (½ in.), and 0.64 cm. (¼ in.); third, determining the percentage by weight retained on each screen; fourth, recording the mechanical analysis in the following manner:

Percentage passing 0.64-cm. (¼-in.) screen.......................... =
Percentage passing 1.27-cm. (½-in.) screen and retained on 0.64-cm. (¼-in.) screen.......................... =
Percentage passing 1.90-cm. (¾-in.) screen and retained on 1.27-cm. (½-in.) screen.......................... =
Percentage passing 2.54-cm. (1-in.) screen and retained on 1.90-cm. (¾-in.) screen.......................... =
.... =

100.00

Mechanical Analysis of Sand or Other Fine Highway Material.

The method shall consist of: First, drying at not more than 110° cent. (230° Fahr.) to a constant weight a sample weighing 50 grammes; second, passing the sample through each of the following mesh sieves, the sieves to be used in the order named:

Meshes per linear inch (2.54 cm.).	Diameter of Wire— Inches.	Millimeters.
10	0.027	0.6858
20	0.0165	0.4191
30	0.01375	0.34925
40	0.01025	0.26035
50	0.009	0.22865
80	0.00575	0.1460
100	0.0045	0.1143
200	0.00235	0.05969

third, determining the percentage by weight retained on each sieve, the sifting being continued on each sieve until less than 1% of the weight retained on each sieve shall pass through the sieve during the last minute of sifting; fourth, recording the mechanical analysis in the following manner:

Percentage passing 200-mesh sieve.......................... =
Percentage passing 100-mesh sieve and retained on 200-mesh sieve.......................... =
Percentage passing 80-mesh sieve and retained on 100-mesh sieve.......................... =
Percentage passing 50-mesh sieve and retained on 80-mesh sieve.......................... =
.... =

100.00

MECHANICAL ANALYSIS OF MIXTURES OF SAND OR OTHER FINE HIGHWAY MATERIAL WITH BROKEN STONE, BROKEN SLAG, OR GRAVEL.

The method shall consist of: First, drying at not more than 110° cent. (230° Fahr.) to a constant weight, a sample weighing in pounds six times the diameter in inches of the largest holes required; second, separating the sample by the use of a 10-mesh sieve (American Society for Testing Materials standard sieve); third, examining the portion retained on the 10-mesh sieve in accordance with the method for making a "Mechanical Analysis of Broken Stone, Broken Slag, or Gravel"; fourth, examining the portion passing the 10-mesh sieve in accordance with the method for making a "Mechanical Analysis of Sand or Other Fine Highway Material"; fifth, recording the mechanical analysis in the following manner:

Percentage passing 200-mesh sieve........................ =
Percentage passing 100-mesh sieve and retained on 200-mesh sieve .... =
Percentage passing 80-mesh sieve and retained on 100-mesh sieve .... =
.... =
Percentage passing 10-mesh sieve and retained on 20-mesh sieve .... =
Percentage passing 0.64-cm. (¼-in.) screen and retained on 10-mesh sieve.................................. =
Percentage passing 1.27-cm. (½-in.) screen and retained on 0.64-cm. (¼-in.) screen.......................... =
Percentage passing 1.90-cm. (¾-in.) screen and retained on 1.27-cm. (½-in.) screen.......................... =
.... =
.... = 100.00

VOIDS IN MINERAL AGGREGATES.*

"The voids in mineral aggregates shall be determined by the Cone Specific Gravity Method. In the method of making the determination of voids, as hereinafter described, there shall be used a truncated cone made of No. 18, B. & S.-gauge galvanized steel with caulked seams, and having the following dimensions: over-all diameter of bottom, 25.4 cm. (10 in.); over-all height, 25.4 cm. (10 in.); inside diameter of opening, 7.6 cm. (3 in.). The test shall be made in the following manner: First, thoroughly mix the aggregate by rolling on paper; second, fill the cone with aggregates, avoiding segregation; third, compact aggregate in cone by oscillation on edge of cone resting on wooden floor, wooden box, or block of wood, and use cotton waste pressed against surface of aggregate to prevent segregation during oscillation;

* Proposed in 1915 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

fourth, continue to add aggregate and compact until the cone is full of thoroughly compacted aggregate, which process will require from 300 to 500 oscillations; fifth, weigh cone with aggregate; sixth, weigh cone empty; seventh, weigh cone full of clean water; eighth, determine the specific gravity of aggregate; ninth, the percentage of voids in the aggregate shall be calculated by the following formula:

$$\text{Percentage of voids} = \left(1 - \frac{C - A}{(B - A)\ D}\right) 100$$

in which A = the weight in grammes of the cone; B = the weight in grammes of the cone filled with water; C = the weight in grammes of the cone filled with compacted aggregate; D = the specific gravity of the aggregate."

Rattler Test for Paving Brick.*

Construction of the Rattler.

General Design.—The machine shall be of good mechanical construction, self-contained, shall conform to the following details of material and dimensions, and shall consist of barrel, frame, and driving mechanisms as herein described.

The Barrel.—The barrel of the machine shall be made up of the heads, head-liners, staves, and stave-liners.

The Frame and Driving Mechanism.—The barrel shall be mounted on a cast-iron frame of sufficient strength and rigidity to support it without undue vibration. It shall rest on a rigid foundation with or without the interposition of wooden plates, and shall be fastened thereto by bolts at not less than four points. It shall be driven by gearing having a ratio of driver to driven of not less than one to four.

The Abrasive Charge.—The abrasive charge shall consist of cast-iron spheres of two sizes. When new, the larger spheres shall be 9.52 cm. (3.75 in.) in diameter and shall weigh approximately 3.40 kg. (7.5 lb.) each. Ten spheres of this size shall be used. These shall be weighed separately after each ten tests, and if the weight of any large sphere falls to 3.175 kg. (7 lb.), it shall be discarded and a new one substituted; provided, however, that all the large spheres shall not be discarded and substituted by new ones at any single time, and that, so far as possible, the large spheres shall compose a graduated series in various stages of wear. When new, the smaller spheres shall be 4.762 cm. (1.875 in.) in diameter and shall weigh approximately 0.43 kg. (0.95 lb.) each. In general, the number of small spheres in a charge shall not fall below 245 nor exceed 260. The collective weight of the large and small spheres shall be as nearly 136 kg. (300 lb.) as possible. No small sphere shall be retained in use after it has been worn down so that it will pass a circular hole 4.45 cm. (1.75 in.) in diameter, drilled in an iron plate 0.64 cm. (¼ in.) in thickness, or weigh less than

* Adapted from the "Standard Specifications for Paving Brick", adopted in 1915 by the Am. Soc. for Testing Materials.

0.34 kg. (0.75 lb.). Further, the small spheres shall be tested, by passing them over the above plate or by weighing, after ten tests, and any which pass through or fall below the specified weight, shall be replaced by new spheres; provided, further, that all the small spheres shall not be rejected and replaced by new ones at any one time, and that, so far as possible, the small spheres shall compose a graduated series in various stages of wear. At any time that any sphere is found to be broken or defective, it shall at once be replaced.

The iron composing these spheres shall have a chemical composition within the following limits:

Combined carbon	Not less than	2.50	per	cent.
Graphitic carbon	Not more than	0.25	"	"
Silicon	" " "	1.00	"	"
Manganese	" " "	0.50	"	"
Phosphorus	" " "	0.25	"	"
Sulphur	" " "	0.08	"	"

Operation of the Test.

The Brick Charge.—The number of bricks per test shall be ten for all bricks of so-called "block-size", having dimensions which fall between 20.32 and 22.86 cm. (8 and 9 in.) in length, 7.62 and 9.52 cm. (3 and 3¾ in.) in breadth, and 9.52 and 10.8 cm. (3¾ and 4¼ in.) in thickness. No brick should be selected as part of a regular test that would be rejected by any other requirements of the specifications under which the purchase is made. (*Note by Committee.*—Each brick should be marked by small holes drilled in one of the faces of the brick, and the initial weight of each brick composing the charge should be determined.)

Speed and Duration of Revolution.—The rattler shall be rotated at a uniform rate of not less than 29.5 nor more than 30.5 rev. per min., and 1 800 revolutions shall constitute the test. A counting machine shall be attached to the rattler for recording the revolutions. A margin of not more than 10 revolutions will be allowed for stopping. Only one start and stop per test is generally acceptable. If, from accidental causes, the rattler is stopped and started more than once during a test, and the loss exceeds the maximum permissible under the specifications, the test shall be discarded and another made.

The Scales.—The scales shall have a capacity of not less than 136 kg. (300 lb.), shall be sensitive to 14.17 grammes (0.5 oz.), and shall be tested by a standard test weight at intervals of not less than every ten tests.

The Results.—The loss shall be calculated in percentage of the initial weight of the brick composing the charge. In weighing the rattled brick, any piece weighing less than 0.45 kg. (1 lb.) shall be

rejected. (*Note by Committee.*—The loss for each brick should also be calculated in percentage of the initial weight of each brick composing the charge.)

Absorption of Water by Wood Blocks After Treatment.

Five blocks of average character shall be heated in an oven to a temperature of 110° cent. (230° Fahr.) for 3 hours, then weighed, and immersed in water for the same length of time. At the end of this time, they shall be taken out, wiped dry, and weighed, the difference in weight before and after immersion, calculated on the weight after heating, being the percentage of absorption.

Cement.

For sampling, analysis, and testing of cement, the methods described in the "Final Report of the Special Committee on Uniform Tests of Cement" (*Transactions*, Am. Soc. C. E., Vol. LXXV, 1912, pp. 665 to 696) are recommended for use.

APPENDIX C

TESTS OF BITUMINOUS MATERIALS.

It is recommended that the following methods for performing tests of bituminous materials be adopted as standards:

SPECIFIC GRAVITY.

For liquid and semi-solid materials, some standard form of pyknometer shall be used. For solid materials, the suspension method shall be used. Material and distilled water shall have a temperature of 25° cent. (77° Fahr.).

The pyknometer to be used shall consist of a fairly heavy, straight-walled glass tube, 70 mm. (2.75 in.) long and 22 mm. (0.875 in.) in diameter, ground to receive a solid glass stopper with a hole of 1.6-mm. (0.063-in.) bore, in place of the usual capillary opening. The lower part of this stopper shall be made concave in order to allow all air bubbles to escape through the bore. The depth of the cup-shaped depression is 4.8 mm. (0.188 in.) at the center. The stoppered tube shall have a capacity of about 24 cu. cm. (0.811 oz.) and when empty shall weigh about 28 grammes. Its principal advantages are: (1) that any desired quantity of bituminous material may be poured in without touching the sides above the level desired; (2) it is easily cleaned; (3) on account of the 1.6-mm. (0.063-in.) bore, the stopper can be more easily inserted when the tube is filled with a very viscous oil than if it contained a capillary opening. When testing solid or semi-solid materials with the pyknometer, extreme care should be taken in melting, to avoid loss by evaporation, and, in filling the pyknometer, to avoid entrapping air.

When working with semi-solid bituminous materials which are too soft to be broken and handled in fragments, the following method of determining their specific gravity has been used with good results. The clean, dry pyknometer is first weighed empty and this weight is called *a*. It is then filled in the usual manner with freshly distilled water at 25° cent. (77° Fahr.), and the weight is again taken and called *b*. A small quantity of the material is then placed in a spoon and brought to a fluid condition by the gentle application of heat, with care that no loss by evaporation occurs. When sufficiently fluid, enough is poured into the dry pyknometer, which may also be warmed, to fill it about half full, without allowing the material to touch the sides of the tube above the desired level. The tube and contents are then allowed to cool to room temperature, after which the tube, with the stopper, is carefully weighed. This weight is called *c*. Distilled water, at 25° cent. (77° Fahr.), is then poured in until

the pyknometer is full. After this the stopper is inserted and the whole is cooled to 25° cent. (77° Fahr.) by a 30-min. immersion in a beaker of distilled water maintained at this temperature. All surplus moisture is then removed with a soft cloth, and the pyknometer and contents are weighed. This weight is called *d*. From the weights obtained, the specific gravity of the material may be readily calculated by the following formula:

$$\text{Specific gravity } \frac{25^\circ \text{ cent. } (77^\circ \text{ Fahr.})}{25^\circ \text{ cent. } (77^\circ \text{ Fahr.})} = \frac{c - a}{(b - a) - (d - c)}.$$

Both *a* and *b* are constants, and need be determined only once. It is necessary, therefore, to make only two weighings for each determination after the first. Results obtained according to this method are accurate to within two units in the third decimal place, whereas the open-tube method commonly used is accurate to the second decimal place only.

The specific gravity of fluid bituminous material may be determined in the ordinary manner with this pyknometer by completely filling it with the material and dividing the weight of the bituminous material thus obtained by that of the same volume of water.

Flash Point.

The flash point shall be determined by the closed-cup test.

Although, for ordinary purposes, the open-cup method of determining the flash and burning points of bituminous materials is reasonably accurate, the closed-cup method described below is to be preferred.

The oil tester shall consist of a copper oil cup having a capacity of about 300 cu. cm. (10.1 oz.), and shall be heated in a water or oil bath by a small Bunsen flame. The cup shall be provided with a glass cover carrying a thermometer, and a hole for inserting the testing flame. The testing flame shall be obtained from a jet of gas passed through the piece of glass tubing, and shall be about 5 mm. (0.197 in.) in length.

The flash test shall be made as follows: The oil cup shall first be removed and the bath filled with water or cottonseed oil. The oil may always be used, and is necessary for bituminous material flashing at a temperature of more than 100° cent. (212° Fahr.). The oil cup shall be replaced and filled with the material to be tested to within 3 mm. (0.118 in.) of the flange joining the cup and the vapor chamber above. The glass cover shall then be placed on the oil cup and the thermometer adjusted so that its bulk shall be just covered by the bituminous material. The Bunsen flame shall be applied in such a manner that the temperature of the material in the

cup shall be raised at the rate of about 5° cent. (9° Fahr.) per min. From time to time the testing flame shall be inserted in the opening in the cover to about half way between the surface of the material and the cover. The appearance of a faint bluish flame over the entire surface of the bituminous material will show that the flash point has been reached and the temperature at this point is taken.

SOLUBILITY IN CARBON DISULPHIDE (CS_2).

This test shall consist in dissolving the bituminous material in carbon disulphide and recovering any insoluble matter by filtering the solution through an asbestos felt. The Gooch crucible used for the determination shall be 4.4 cm. (1.722 in.) wide at the top, tapering to 3.6 cm. (1.417 in.) at the bottom, and shall be 2.5 cm. (0.984 in.) deep.

The asbestos shall be cut with scissors into pieces not exceeding 1 cm. (0.394 in.) in length, after which it shall be shaken up with just sufficient water to pour easily. The crucible shall be filled with the suspended asbestos and allowed to settle for a few moments. A light suction shall then be applied to draw off all the water and leave a firm mat of asbestos in the crucible. More of the suspended material shall be added, and the operation shall be repeated until the felt shall be so dense that it scarcely transmits light when held so that the bottom of the crucible is between the eye and the source of light. The felt shall then be washed several times with water, and drawn firmly against the bottom of the crucible by an increased suction. The crucible shall be removed to a drying oven for a few minutes, after which it shall be ignited at red heat over a Bunsen burner, cooled in a desiccator, and weighed.

Two grammes of bituminous material or 10 grammes of an asphalt topping or rock asphalt shall then be placed in an Erlenmeyer flask, which shall have been weighed previously, and the accurate weight of the sample obtained. One hundred cubic centimeters (3,381 oz.) of chemically pure carbon disulphide shall be poured into the flask, in small portions, with continual agitation, until all lumps disappear and nothing adheres to the bottom. The flask shall then be corked and set aside for 15 min. to allow settlement of the insoluble material.

The weighed Gooch crucible containing the felt shall be set up over the dry pressure flask, and the solution of bituminous material in carbon disulphide shall be decanted through the felt without suction by gradually tilting the flask, with care not to stir up any precipitate that may have settled out. At the first sign of any sediment coming out, the decantation shall be stopped and the filter allowed to drain. A small quantity of carbon disulphide shall then be washed down the sides of the flask, after which the precipitate shall be brought upon the felt and the flask scrubbed, if necessary,

with a feather or "policeman", to remove all adhering material. The contents of the crucible shall be washed with carbon disulphide until the washings run colorless. Suction shall then be applied until there is practically no odor of carbon disulphide in the crucible, after which the outside of the crucible shall be cleaned with a small quantity of the solvent. The crucible and contents shall be dried in the hot-air oven at 100° cent. (212° Fahr.) for about 20 min., cooled in a desiccator, and weighed. If any appreciable quantity of insoluble matter adheres to the flask, it shall also be dried and weighed, and any increase over the original weight of the flask shall be added to the weight of insoluble matter in the crucible. The total weight of insoluble material may include both organic and mineral matter. The former, if present, shall be burned off by ignition at a red heat until no incandescent particles remain, thus leaving the mineral matter or ash, which can be weighed on cooling. The difference between the total weight of material insoluble in carbon disulphide and the weight of substance taken equals the total bitumen, and the percentage weights are calculated and reported as total bitumen, and insoluble organic and inorganic matter, on the basis of the weight of material taken for analysis.

This method is quite satisfactory for straight oil and tar products, but, where native asphalts are present, it will be found practically impossible to retain all the finely divided mineral matter on an asbestos felt. It is generally more accurate, therefore, to obtain the result for total mineral matter by direct ignition of a 1-gramme sample in a platinum crucible, or to use the result for ash obtained in the fixed carbon test. The total bitumen is then determined by deducting from 100% the sum of the percentages of total mineral matter and insoluble organic matter. If the presence of a carbonate mineral is suspected, the percentage of mineral matter may be most accurately obtained by treating the ash from the fixed carbon determination with a few drops of ammonium carbonate solution, drying at 100° cent. (212° Fahr.), then heating for a few minutes at a dull red heat, cooling, and weighing again.

When difficulty in filtering is experienced—for instance, when Trinidad asphalt is present in any quantity—a longer period of subsidence than 15 min. is necessary, and the following method, adopted in 1911 by the American Society for Testing Materials is recommended:

Analysis of Sample.—After drying, from 2 to 15 grammes (as may be necessary to insure the presence of 1 to 2 grammes of pure bitumen) are weighed into a 150-cc. tared Erlenmeyer flask, and treated with 100 cc. of carbon disulphide. The flask is then loosely corked and shaken from time to time until all large particles of the material have been broken up. It is then set aside for 48 hours to settle. The

solution is decanted into a similar flask that has been previously weighed. As much of the solvent is poured off as possible without disturbing the residue. The contents of the first flask are again treated with fresh carbon disulphide, shaken as before, and then put away with the second flask for 48 hours to settle.

The liquid in the second flask is then carefully decanted on a weighed Gooch crucible, 3.2 cm. in diameter at the bottom, fitted with an asbestos filter, and the contents of the first flask are similarly treated. The asbestos filter is made of ignited long-fiber amphibole, packed in the bottom of a Gooch crucible to the depth of not more than ⅛ in. In filtering, no vacuum is to be used and the temperature is to be kept between 20° and 25° cent. After passing the liquid contents of both flasks through the filter, the residue on the filter is thoroughly washed, and the residues remaining in them are shaken with more fresh carbon disulphide and allowed to settle for 24 hours, or until it is seen that a good subsidation has taken place. The solvent in both flasks is then again decanted through the filter, and the residues remaining in them are washed until the washings are practically colorless. All washings are to be passed through the Gooch crucible.

The crucible and both flasks are then dried at 125° cent. and weighed. The filtrate containing the bitumen is evaporated, the bituminous residue burned, and the weight of the ash thus obtained added to that of the residue in the two flasks and the crucible. The sum of these weights deducted from the weight of substance taken gives the weight of soluble bitumen.

Solubility of Bitumen in Carbon Tetrachloride (CCl_4).

The test shall be conducted in exactly the same manner as described for the test for "Solubility in Carbon Disulphide", except that 100 cu. cm. (3.381 oz.) of chemically pure carbon tetrachloride shall be used in place of carbon disulphide, and the percentage of bitumen insoluble in carbon tetrachloride shall be reported on the basis of the bitumen taken as 100, the quantity of bitumen having been determined by the method described under the heading "Solubility in Carbon Disulphide."

Consistency.

The "Engler Viscosimeter," the "New York Testing Laboratory Float," or the "Penetrometer," shall be used, as practicable, at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), and 46° cent. (115° Fahr.).

Viscosity Test.

The viscosity of liquid bituminous materials shall be determined at any desired temperature by using the "Engler Viscosimeter." This apparatus consists of a brass vessel for holding the material to be

tested, and is closed by a cover. To the conical bottom is fitted a conical outflow tube exactly 20 mm. (0.787 in.) long, with a diameter of 2.9 mm. (0.114 in.) on top, and of 2.8 mm. (0.110 in.) on the bottom. This tube is closed and opened by a pointed hardwood stopper. Pointed metal projections are placed on the inside of the vessel at equal distances from the bottom, and serve for measuring the charge of material, which is 240 cu. cm. (8.116 oz.). A thermometer is used to ascertain the temperature of the material to be tested. The vessel is surrounded by a brass jacket, which holds the material which may be used as a heating bath, either water or cottonseed oil, according to the temperature at which the test is to be made. A tripod serves as a support for the apparatus, and also carries a ring burner by which the bath is heated directly. The measuring cylinder, having a capacity of 100 cu. cm. (3.381 oz.), which is sufficiently accurate for work with road materials, is placed directly under the outflow tube.

As all viscosity determinations should be compared with that of water at 25° cent. (77° Fahr.), the apparatus shall have been previously calibrated as follows: The cup and outlet tube shall first be scrupulously cleaned. A piece of soft tissue paper is convenient for cleaning the tube. The stopper shall then be inserted in the tube, and the cup shall be filled with water at 25° cent. (77° Fahr.) to the top of the projections. The measuring cylinder shall be placed directly under the outflow tube so that the material, on flowing out, will not touch the sides. The stopper shall then be removed and the time required, both for 50 and 100 cu. cm. (1.691 and 3.381 oz.) to run out, shall be ascertained by using a stop-watch. The results thus obtained shall be checked a number of times. The time required for 50 cu. cm. (1.691 oz.) of water should be about 11 sec., and for 100 cu. cm. (3.381 oz.) about 22.8 sec.

Bituminous materials shall be tested in the same manner as water, and the temperature at which the test is made shall be controlled by the bath. The material shall be brought to the desired temperature and maintained there for at least 3 min. before making the test. The results are expressed as specific viscosity compared with water at 25° cent. (77° Fahr.), as follows:

Specific viscosity at —— degrees centigrade for —— cu. cm.

$$= \frac{\text{second for passage of given volume at —— degrees centigrade}}{\text{seconds for passage of same volume of water at 25° cent. (77° Fahr.)}}$$

Float Test.

The float apparatus consists of two parts, an aluminum float or saucer and a conical brass collar. The two parts are made separately, so that one float may be used with a number of brass collars.

In making the test, the brass collar shall be placed with the small end down on the brass plate, which shall have been previously amalgamated with mercury by rubbing it first with a dilute solution of mercuric chloride or nitrate and then with mercury. A small quantity of the material to be tested shall be heated in the metal spoon until quite fluid, with care that it shall suffer no appreciable loss by volatilization and that it shall be kept free from air bubbles. It shall then be poured into the collar in a thin stream until slightly more than level with the top. After the material has cooled to room temperature, the surplus may be removed with a spatula blade which has been slightly heated. The collar and plate shall then be placed in one of the tin cups containing ice water maintained at 5° cent. (41° Fahr.), and left in this bath for 15 min. Meanwhile, the other cup shall be filled about three-fourths full of water and placed on the tripod, and the water shall be heated to any temperature desired for the test. This temperature shall be accurately maintained, and shall at no time throughout the entire test be allowed to vary more than 0.5° cent. (0.9° Fahr.) from the temperature selected. After the material to be tested has been kept in the ice water for 15 min., the collar and contents shall be removed from the plate and screwed into the aluminum float, which shall then be immediately floated in the warmed bath. As the plug of bituminous material becomes warm and fluid, it is gradually forced upward and out of the collar, until water gains entrance to the saucer and causes it to sink.

The time, in seconds, between placing the apparatus on the water and when the float sinks shall be taken as a measure of the consistency of the material under examination.

Penetration Test.*

Apparatus.—The container for holding the material to be tested shall be a flat-bottomed, cylindrical dish, 55 mm. ($2\frac{3}{16}$ in.) in diameter and 35 mm. ($1\frac{3}{8}$ in.) deep.* The needle for this test shall be a cylindrical steel rod 50.8 mm. (2 in.) long and having a diameter of 1.016 mm. (0.04 in.) and turned on one end to a sharp point having a 6.35-mm. ($\frac{1}{4}$-in.) taper. The water bath shall be maintained at a temperature not varying more than 0.1° cent. (0.18° Fahr.) from 25° cent. (77° Fahr.). The volume of water shall be not less than 10 liters, and the sample shall be immersed to a depth of not less than 10 cm. (4 in.) and shall be supported on a perforated shelf not less than 5 cm. (2 in.) from the bottom of the bath. Any apparatus which will allow the needle to penetrate without appreciable friction, and which is accurately calibrated to yield results in accordance with the definition of penetration, will be acceptable. The transfer dish for the container shall be a small dish or tray of such capacity as will

* Adopted in 1916 by the Am. Soc. for Testing Materials.

insure complete immersion of the container during the test. It shall be provided with some means which will insure a firm bearing and prevent rocking of the container.

Preparation of Sample.—The sample shall be completely melted at the lowest possible temperature, and stirred thoroughly until it is homogeneous and free from air bubbles. It shall then be poured into the sample container to a depth of not less than 15 mm. (⅝ in.). The sample shall be protected from dust and allowed to cool in an atmosphere not lower than 18° cent. (65° Fahr.) for 1 hour. It shall then be placed in the water bath along with the transfer dish and allowed to remain 1 hour.

Testing.—In making the test, the sample shall be placed in the transfer dish filled with water from the water bath of sufficient depth to cover the container completely. The transfer dish containing the sample shall then be placed on the stand of the penetration machine. The needle, loaded with specified weight (See Report Form for Asphalt Cement, Appendix A), shall be adjusted to make contact with the surface of the sample. This may be accomplished by making contact of the actual needle point with its image reflected by the surface of the sample from a properly placed source of light. Either the reading of the dial shall then be noted or the needle brought to zero. The needle is then released for the specified period of time, after which the penetration machine is adjusted to measure the distance penetrated. At least three tests shall be made at points on the surface of the sample not less than 1 cm. (⅜ in.) from the side of the container and not less than 1 cm. (⅜ in.) apart. After each test the sample and transfer dish shall be returned to the water bath, and the needle shall be carefully wiped toward its point with a clean, dry cloth, to remove all adhering bitumem. The reported penetration shall be the average of at least three tests the values of which shall not differ more than four points between maximum and minimum. When desirable to vary the temperature, time, and weight (See Report Form for Asphalt Cement, Appendix A), and, in order to provide for a uniform method of reporting results when variations are made, the samples shall be melted and cooled in air as above directed. They shall then be immersed in water or brine, as the case may require, for 1 hour at the temperature desired.

Melting Point.

Cube Method for Tar Cements.

The material under examination shall be first melted in a spoon by the gentle application of heat until sufficiently fluid to pour readily. Care shall be taken that it suffers no appreciable loss by volatilization. It shall then be poured into a 12.7 mm. (0.5-in.) brass cubical mould, which shall have been amalgamated with mercury, and shall be placed

on an amalgamated brass plate. The brass may be amalgamated by washing it first with a dilute solution of mercuric chloride or nitrate, after which the mercury is rubbed into the surface. By this means the bituminous material is, to a considerable extent, prevented from sticking to the sides of the mould. The hot material shall slightly more than fill the mould, and, when cooled, the excess shall be cut off with a hot spatula.

After cooling to room temperature, the cube shall be removed from the mould and fastened on the lower arm of a No. 10 wire (B. & S. gauge), bent at right angles at one end and suspended beside a thermometer in a covered Jena glass beaker having a capacity of 400 cu. cm. (13.526 oz.), which shall be placed in a water bath, or, for high temperatures, a cottonseed-oil bath. The wire shall be passed through the center of two opposite faces of the cube, which shall then be suspended with its base 25.4 mm. (1 in.) above the bottom of the beaker. The water or oil bath shall consist of an 800-cu. cm. (27.051-oz.) low-form Jena glass beaker, suitably mounted for the application of heat from below. The beaker in which the cube is suspended shall be of the tall-form Jena type, without lip. The metal cover shall have two openings. A cork, through which passes the long arm of the wire, shall be inserted in one hole and the thermometer in the other. The bulb of the thermometer shall be just level with the cube and at an equal distance from the side of the beaker. In order that a reading of the thermometer may be made, if necessary, at the point which passes through the cover, the hole shall be triangular and covered with an ordinary object glass through which the stem of the thermometer may be seen. Readings made through this glass shall be calibrated to the angle of observation, which may be made constant by sighting always from the front edge of the opening to any given point on the stem of the thermometer below the cover.

After the test specimen shall have been placed in the apparatus, the liquid in the outer vessel shall be heated in such a manner that the thermometer registers an increase of 5° cent. (9° Fahr.) per min. The temperature at which the bituminous material touches a piece of paper placed in the bottom of the beaker shall be taken as the melting point. Determinations made in the manner described shall not vary more than 2° cent. (3.6° Fahr.) for successive trials on the same material. At the beginning of this test the temperature of both bituminous material and bath shall be approximately at 25° cent. (77° Fahr.).

Ring and Ball Method for Asphalt Cements.*

The apparatus shall consist of a brass ring, 15.875 mm. ($\frac{5}{8}$ in.) in diameter, 6.35 mm. ($\frac{1}{4}$ in.) deep, 2.38125 mm. ($\frac{3}{32}$ in.) wide, sus-

*Proposed in 1916 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

pended 25.40 mm. (1 in.) above bottom of a beaker; a steel ball, 9.525 mm. (3/8 in.) in diameter, weighing between 3.45 and 3.50 grammes; a standardized thermometer; a glass beaker, approximately 600-cc. capacity.

Carefully melt the sample and fill the ring with material to be tested. Remove any excess. Place ball in center of ring and suspend in beaker containing approximately 400 cc. of water at a temperature of 5° cent. (41° Fahr.). Arrange thermometer bulb within 1/2 in. of sample and at same level. Apply heat uniformly over bottom of beaker in quantity sufficient to raise temperature 5° cent. (9° Fahr.) per min. Record temperature at starting test and every minute thereafter until test is completed. The rate of heating is very important. Softening point is temperature at which specimen has dropped 1 in. Successive tests should average within 3° cent. For temperatures above 95° cent., glycerine shall be used instead of water.

Loss on Evaporation.*

The amount lost by oils and asphaltic compounds when they are heated in an oven at a temperature of 163° cent. (325° Fahr.) plus or minus 1° cent. (2° Fahr.) shall be determined by heating 50 grammes of the water-free substance contained in a flat-bottomed dish, the inside dimensions of which are approximately 2 3/16 in. in diameter and 1 3/8 in. deep (3-oz. Gill style ointment box, deep style) for 5 hours. The oven in which the substance is heated shall be brought to the prescribed temperature before the sample is introduced, and the temperature of the sample under test shall be regarded as that of a similar quantity of the same material immediately adjoining it in the oven, in which the bulb of a standardized thermometer is immersed. The oven may be either of circular or rectangular form, and the source of heat either gas or electricity. The samples under test shall rest in the same relative position in a single row on a perforated circular shelf, 9 1/4 in. in diameter, suspended by a vertical shaft midway in the oven, which is revolved by mechanical means at the rate of from 5 to 6 rev. per min. (Note.—If additional periods of heating are desired, it is recommended that they be made in successive increments of 5 hours each.) If the residue after heating is to be tested for penetration, the sample should be thoroughly mixed by stirring until it is cool, and thereafter manipulated in accordance with the directions of the standard test for penetration of bituminous materials.

Distillation.*

Note.—Equivalents in English units have been added by the Special Committee on Materials for Road Construction.

* Adopted in 1916 by the Am. Soc. for Testing Materials.

Sampling.—The sample as received shall be thoroughly stirred and agitated, warming, if necessary, to insure a complete mixture before the portion for analysis is removed.

Dehydration.—If the presence of water is suspected, or known, the material shall be dehydrated before distillation. About 500 cu. cm. (16.907 oz.) of the material is placed in an 800-cu. cm. (27.051-oz.) copper still provided with a distilling head connected with a water-cooled condenser. A ring burner is used, starting with a small flame at the top of the still, and gradually lowering it, if necessary, until all the water has been driven off. The distillate is collected in a 200-cu. cm. separatory funnel with the tube cut off close to the stop-cock. When all the water has been driven over and the distillate has settled out, the water is drawn off and the oils are returned to the residue in the still. The contents of the still shall have cooled to below 100° cent. (212° Fahr.) before the oils are returned, and they shall be well stirred and mixed with the residue.

Apparatus.—The apparatus shall consist of the following standard parts:

(*a*) *Flask.*—The distillation flask shall be a 250-cu. cm. Engler distilling flask, having the following dimensions:

Diameter of bulb........................	8.0 cm.	(3.150 in.)
Length of neck........................	15.0 "	(5.906 ")
Diameter of neck........................	1.7 "	(0.670 ")
Surface of material to lower side of tubulature.	11.0 "	(4.331 ")
Length of tubulature........................	15.0 "	(5.906 ")
Diameter of tubulature........................	0.9 "	(0.354 ")
Angle of tubulature........................	75°	

A variation of 3% from the foregoing measurements will be allowed.

(*b*) *Thermometer.*—The thermometer shall conform to the following requirements:

It shall be made of thermometric glass of a quality equivalent to suitable grades of Jena or Corning make. It shall be thoroughly annealed. It shall be filled above the mercury with inert gas which will not act chemically on or contaminate the mercury. The pressure of the gas shall be sufficient to prevent separation of the mercury column at all temperatures of the scale. There shall be a reservoir above the final graduation large enough so that the pressure will not become excessive at the highest temperature. The thermometer shall be finished at the top with a small glass ring or button suitable for attaching a tag. Each thermometer shall have for identification the maker's name, a serial number, and the letters "A. S. T. M. Distillation."

The thermometer shall be graduated from 0 to 400° cent. at intervals of 1° cent. Every fifth graduation shall be longer than the inter-

mediate ones, and every tenth graduation beginning at zero shall be numbered. The graduation marks and numbers shall be clear-cut and distinct.

The thermometer shall conform to the following dimensions:

Total length, maximum......	385 mm.	
Diameter of stem............	7 "	; permissible variation, 0.5 mm.
Diameter of bulb, minimum..	5 "	; and shall not exceed diameter of stem.
Length of bulb............	12.5 "	permissible variation, 2.5 mm.
Distance from 0° to bottom of bulb	30 "	; " " 5 "
Distance from 0° to 400°...	295 "	; " " 10 "

The accuracy of the thermometer when delivered to the purchaser shall be such that when tested at full immersion the maximum error from 0 to 200° cent. shall not exceed the following:

From 0 to 200° cent....................0.5° cent.
" 200 " 300° cent....................1.0° cent.
" 300 " 375° cent....................1.5° cent.

The sensitiveness of the thermometer shall be such that when cooled to a temperature of 74° cent. below the boiling point of water at the barometric pressure, at the time of test, and plunged into free flow of steam, the meniscus shall pass the point 10° cent. below the boiling point of water in not more than 6 sec.

The thermometer shall be set up as for the distillation test, using water, naphthalene, and benzophenone as distilling liquids. The correctness of the thermometer shall be checked at 0 and 100° cent. after each third distillation until seasoned.

(c) *Condenser.*—The condenser tube shall have the following dimensions:

Length	500 mm.		(19.685 in.)
Width	12 to 15	"	(0.472 to 0.591 in.)
Width of adaptor end...	20 " 25	"	(0.787 " 0.984 ")

(d) *Stands.*—Two iron stands shall be provided, one with a universal clamp for holding the condenser, and one with a light grip arm with a cork-lined clamp for holding the flask.

(e) *Burner and Shield.*—A Bunsen burner shall be provided, with a tin shield, 20 cm. (7.784 in.) long and 9 cm. (3.543 in.) in diameter. The shield shall have a small hole through which to observe the flame.

(f) *Cylinders.*—The cylinders used in collecting the distillate shall have a capacity of 25 cu. cm. (0.845 oz.), and shall be graduated in tenths of a cubic centimeter.

Setting up the Apparatus.—The apparatus shall be set up, the thermometers being placed so that the top of the bulb is opposite the middle of the tubulature. All connections shall be tight.

Method.—One hundred cubic centimeters (3.381 oz.) of the dehydrated material to be tested shall be placed in a tared flask and weighed. After adjusting the thermometer, shield, condenser, etc., the distillation is commenced, the rate being regulated so that 1 cu. cm. (0.034 oz.) passes over every minute. The receiver is changed as the mercury column just passes the fractionating point.

Up	to	110°	cent.	(230° Fahr.)
110° cent.	"	170°	"	(338° ")
170° cent.	"	235°	"	(455° ")
235° cent.	"	270°	"	(518° ")
270° cent.	"	300°	"	(572° ")

To determine the quantity of residue, the flask is weighed again when distillation is complete. During the distillation the condenser tube shall be warmed when necessary, in order to prevent the deposition of any sublimate. The percentages of fraction should be reported, both by weight and by volume.

Ductility.

A briquette of the material to be tested shall be formed by pouring the molten material into a briquette mould. The dimensions of the briquette shall be: 1 cm. (0.394 in.) in thickness throughout its entire length; distance between the clips or end pieces, 3 cm. (1.181 in.); width of asphalt cement section at mouth of clips, 2 cm. (0.787 in.); width at minimum cross-section, half way between clips, 1 cm. (0.394 in.). The center pieces are removable, the briquette mould being held together during moulding with a clamp or wire.

The moulding of the briquette shall be done as follows: The two center sections shall be well amalgamated to prevent the asphalt cement from adhering to them, and the briquette mould shall then be placed on a freshly amalgamated brass plate. The asphalt cement to be tested, while in a molten state, shall be poured into the mould, a slight excess being added to allow for shrinkage on cooling. When the asphalt cement in the mould is nearly cool, the briquette shall be cut off level, with a warm knife or spatula. When it is thoroughly cooled to the temperature at which it is desired to make the test, the clamp and the two side pieces are removed, leaving the briquette of asphalt cement held at each end by the ends of the mould, which now play the part of clips. The briquette shall be kept in water for 30 min. at 4° cent. (39° Fahr.) or 25° cent. (77° Fahr.) before testing, dependent on the temperature at which the ductility is desired. The briquette with the clips attached shall then be placed in a "ductility test machine" filled

with water at one of the above temperatures to a sufficient height to cover the briquette not less than 50 mm. (1.969 in.). This machine consists of a rectangular water-tight box, having a movable block working on a worm-gear from left to right. The left clip is held rigid by placing its ring over a short metal peg provided for this purpose; the right clip is placed over a similar rigid peg on the movable block. The movable block is provided with a pointer which moves along a centimeter scale. Before starting the test, the centimeter scale is adjusted to the pointer at zero. Power is then applied by the worm-gear pulling from left to right at the uniform rate of 5 cm. (1.969 in.) per min. The distance, in centimeters, registered by the pointer on the scale at the time of rupture of the thread of asphalt cement shall be taken as the ductility of the asphalt cement.

Solubility in Petroleum Naphtha.

Two grammes of the material shall be placed in a 4-oz. oil-sample bottle, made up to 100 cu. cm. (3.381 oz.) with 88° Baumé petroleum naphtha (boiling point between 40° cent. (104° Fahr.) and 55° cent. (131° Fahr.)), and the whole well shaken until the sample is digested. The bottle shall then be centrifugalized for 10 min., 50 cu. cm. (1.691 oz.) withdrawn into a weighed flask, the naphtha distilled off by a water bath, and the residue weighed. From this weight the percentage of solubility shall be calculated.

Fixed Carbon.

One gramme of the bituminous material shall be placed in a platinum crucible, weighing between 20 and 30 grammes, between 28 and 38 mm. (1.102 and 1.496 in.) in height, and having a tightly fitting cover provided with a flange, about 4 mm. (0.157 in.) in depth. The crucible and its contents shall then be heated, first gently and then more severely, until no smoke or flame shall issue between the crucible and the lid. It shall then be placed in the full flame of a Bunsen burner for 7 min., holding the cover down with the end of a pair of tongs until the most volatile products shall have been burned off. The crucible shall be supported on a platinum triangle with the bottom from 6 to 8 cm. (2.362 to 3.150 in.) above the top of the burner. The flame shall be fully 20 cm. (7.874 in.) high when burning free, and the determination shall be made in a place free from drafts. The upper surface of the cover shall burn clear, but the under surface may or may not be covered with carbon, dependent on the character of the bituminous material. The crucible shall be removed to the desiccator, and, when cool, shall be weighed, after which the cover shall be removed and the crucible placed in an inclined position over the Bunsen burner and ignited until nothing but ash remains. Any carbon deposited on the cover shall also be burned off. The weight of ash

remaining shall be deducted from the weight of the residue after the first ignition of the sample. The resulting weight is that of the fixed carbon, which shall be calculated on the basis of the total weight of the sample, exclusive of mineral matter.

PARAFFIN.

One hundred grammes of the material shall be distilled rapidly in a retort to a dry coke. Five grammes of the distillate shall then be thoroughly mixed in a 60-cu. cm. (2.029-oz.) flask with 25 cu. cm. (0.845 oz.) of Squibbs' absolute ether. Twenty-five cu. cm. (0.845 oz.) of Squibbs' absolute alcohol shall then be added, and the flask packed closely in a freezing mixture of finely crushed ice and salt for at least 30 min. The precipitate shall be filtered out quickly with a suction pump, using a No. 575 C. S. and S. 9-cm. hardened filter paper. The flask and precipitate shall then be rinsed and washed with a mixture of equal parts of Squibbs' alcohol and ether cooled to — 17° cent. (1° Fahr.) until free from oil (50 cu. cm. (1.691 oz.) of washing solution is usually sufficient). When sucked dry, the filter paper shall be removed and the waxy precipitate transferred to a small glass disk and evaporated on a steam bath. The residue (paraffin) remaining on the disk shall be weighed, and from this weight the percentage on the original 5-gramme sample shall be calculated.

SPECIFIC GRAVITY AT 38° CENT. OF WOOD BLOCK PRESERVATIVE.*

A standardized hydrometer shall be used. A set of two with ranges 1.00 to 1.08, and 1.07 to 1.15 will suffice. Before taking the specific gravity, the oil in the cylinder should be stirred thoroughly with a glass rod, and this rod when withdrawn from the liquid should show no solid particles at the instant of withdrawal. Care should be taken that the hydrometer does not touch the sides or bottom of the cylinder when the reading is taken, and that the oil surface is free from froth and bubbles. If the specific gravity is determined at a higher temperature than desired, correction should be made by adding 0.0008 to the reading for each degree centigrade excess of temperature.

SOLUBILITY IN BENZOL OR CHLOROFORM OF WOOD BLOCK PRESERVATIVE.

From 5 to 10 grammes of the water-free oil is weighed out into a weighed 100-c.c. (3.38-oz.) beaker; 50 c.c. (1.69 oz.) of the solvent is added, and the solution is passed through a weighted 9-cm., C. S. and S., No. 575 filter paper in a short-stemmed funnel, the filtrate being passed into the flask to be subsequently used for the hot extraction. The beaker is washed clean from all soluble matter, dried, and weighed. The funnel, with filter paper and contents, is then placed in a N. Y. T. L. or Underwriter's form of glass extraction apparatus, and heat

* Modification of method proposed in 1915 by Committee D-7 on "Standard Specifications for Timber", of the Am. Soc. for Testing Materials.

is applied from a water-bath or hot plate until the extraction is complete and the filtrate runs through colorless. The filter and contents are then dried and weighed. The increase in weight is added to the increase in weight of the beaker, if any, the result being the weight of the insoluble matter. The weight of the insoluble matter thus found is subtracted from the weight of the material taken for analysis. The difference in weight is the weight of the soluble matter, from which the percentage is calculated.

Water Content of Wood Block Preservative.

From 250 to 300 c.c. (8.45 to 10.14 oz.) of the oil is weighed out into a 500-c.c. glass retort, or into a small copper still, provided with a distilling head. Heat is applied with a ring burner, starting with a small flame at the top of the still, and gradually lowering it until all the water has been driven off. The distillate of oil and water is collected in a graduated separatory funnel, the volume of water, in cubic centimeters, is read, and its percentage computed by volume. The water is then drawn off and the oils are returned to the residue in the still. The contents of the still shall have cooled to below 100° cent. (212° Fahr.) before the oils are returned, and they shall be well stirred and mixed with the residue.

Distillation Test for Wood Block Preservative.*

Apparatus for Distillation Test.

Retort.—This shall be a tabulated Jena glass retort of the usual form, with a capacity of 250 to 290 c.c. (8.45 to 9.8 oz.). The capacity shall be measured by placing the retort with the bottom of the bulb and the end of the offtake in the same horizontal plane, and pouring water into the bulb through the tubulature until it overflows the offtake. The quantity remaining in the bulb shall be considered as its capacity.

Shield.—An asbestos shield shall be used to protect the retort from air currents and to prevent radiation. This may be covered with galvanized iron, as such an arrangement is more convenient and more permanent.

Receivers.—Erlenmeyer flasks of from 50 to 100 c.c. capacity are of the most convenient form.

Thermometer.—The thermometer shall be of glass, well annealed, and shall undergo no serious change at the zero point when heated up to 400° cent. The space above the mercury column shall be filled with gas, either carbon dioxide or nitrogen, and the thermometer shall have an expansion chamber at the top. The scale shall read from 0 to 400° cent., in graduations of 1° cent., which shall be etched on the stem. The tip of the thermometer shall carry a ring for the

* Modification of method proposed in 1915 by Committee D-7 on "Standard Specifications for Timber", of the Am. Soc. for Testing Materials.

purpose of attaching tags. The thermometer shall have the following dimensions:

Total length, 375 mm.; tolerance, 10 mm.
Bulb length, 14 mm.; tolerance, 1 mm.
Distance from zero mark to bottom of bulb, 30 mm.; tolerance, 4 mm.
Scale length from zero mark to 400° cent., 295 mm.; tolerance, 5 mm.
Diameter of stem, 7 mm.; tolerance, 1 mm.
Diameter of bulb, 6 mm.; tolerance, 1 mm.

When standardized, the accuracy of such standardization should be as follows:

Up to 200° cent.	to the nearest 0.5° cent.
200 to 300° "	" " " 1.0° "
300 to 360° "	" " " 1.5° "

Assembling for Distillation Test.

The retort shall be supported on a tripod or rings over two sheets of 20-mesh gauge, 15.24 cm. (6 in.) square. It shall be connected to the condenser tube by a tight cork joint. The thermometer shall be inserted through a cork in the tubulature, with the bottom of the bulb 1.27 cm. ($\frac{1}{2}$ in.) from the surface of the oil in the retort. The exact location of the thermometer bulb shall be determined by placing a vertical rule, graduated in divisions not exceeding 0.16 cm. ($\frac{1}{16}$ in.), back of the retort when the latter is in position for the test, and sighting the level of the liquid and the point for the bottom of the thermometer bulb. The distance from the bulb of the thermometer to the outlet end of the condenser tube shall be not more than 60.96 cm. (24 in.) nor less than 50.8 cm. (20 in.). The burner should be protected from drafts by a suitable shield or chimney.

Distillation Test.

Exactly 100 grammes of oil shall be weighed into the retort, the apparatus shall be assembled, and heat applied. The distillation shall be conducted at the rate of at least one drop, and not more than two drops, per second, and the distillate collected in weighed receivers. The condenser tube shall be warmed whenever necessary, to prevent accumulation of solid distillates. Fractions shall be collected at the following points:

Up to 170° cent. (338° Fahr.), 170-200° cent. (338-392° Fahr.), 200-210° cent. (392-410° Fahr.), 210-235° cent. (410-455° Fahr.), 235-270° cent. (455-518° Fahr.), 270-300° cent. (518-572° Fahr.), 300-315° cent. (572-599° Fahr.), 315-355° cent. (599-671° Fahr.).

The receivers shall be changed as the mercury passes the dividing temperature for each fraction. The last receiver shall be removed at

355° cent. (671° Fahr.), and drainage from the condenser, etc., shall not be considered as part of the fraction. For weighing the receivers and fractions, a balance accurate to at least 0.05 gramme shall be used. During the progress of the distillation the thermometer shall remain in its original position. No correction shall be made for the emergent stem of the thermometer.

When any measurable quantity of water is present in the distillate, it shall be separated as nearly as possible and reported separately, all results being calculated on a basis of dry oil. When more than 2% of water is present, water-free oil shall be obtained by separately distilling a larger quantity, returning any oil carried over with the water, and using dried oil for the final distillation. A copper tar still is a convenient implement for obtaining water-free oil.

DISCUSSION

Mr. Gilman.

E. DOW GILMAN,* ASSOC. M. AM. SOC. C. E. (by letter).—Referring to page 1438, on the subject "Abrasion Test for Broken Stone or Broken Slag", the writer notes that the specifications for this test as adopted in 1908 by the American Society for Testing Materials have been adopted in the report of the Committee. These specifications provide that the sample to be tested shall consist, as nearly as possible, of 50 pieces of the broken stone, the total weight of which shall be within 10 grammes of 5 kilogrammes. Under the conditions thus stated, the sample may consist of broken stone the individual pieces of which may be of widely varying size and weight. That a sample composed of 50 pieces of nearly uniform size will give different results from those obtained with a sample composed of 50 pieces some of which are small and others large would seem to require no proof.

Several years ago the writer made abrasion tests on a shipment of trap rock. An attempt was made to check results by duplicating the tests on other samples taken from the same shipment. Three separate tests were made on as many different samples, each of which was carefully prepared according to the specifications just stated. Such varying results were secured that the writer was not satisfied to submit any one or the average of the French coefficients thus obtained. New samples were prepared, each consisting of 50 pieces of the broken stone and with the added specification that no single piece should weigh less than 90 or more than 110 grammes, and that any piece of a shape noticeably different from the general shape of the entire lot should be eliminated. The total weight of the sample was within 10 grammes of 5 kilogrammes, as before. The results were remarkably more consistent than with the other samples, and were so close together that, considering the unavoidable factors in a test of this nature, one would be warranted in submitting an average as the probable coefficient of the material at hand.

The objection to this added clause is the care required to prepare a sample and the quantity of material required from which to take one. In spite of this objection, the writer is inclined to believe that, if results secured by the abrasion test have any practical significance whatever, the test must be worthy of the slight extra labor involved. It seems to be a question whether the operation of the Deval machine for more than 5 hours and the labor in washing and drying the sample before and after the test are worth the time and energy, if the results of two or more samples of the same material are not reasonably close. Even with the utmost care, the tests for abrasion, hardness, toughness, and cementing value, give results that are rough and that can be

* Washington, D. C.

taken as no more than a preliminary guide in passing judgment on the usefulness of a material. With the limited use that at present is made of the results of these tests, it would be unwise to burden them with too many refinements, but the writer submits the suggestion here made in the belief that greater reliance can be placed on comparative results thus obtained. This fact is apparently acknowledged in the specifications for the abrasion test of gravel, immediately following, which, under Method No. 2, insures, at least to a degree, the uniformity of separate samples. Mr. Gilman.

The writer regrets that the data secured in the tests which he made are not available, so that by actual figures he might show comparisons of the methods; but he believes that the point in question can be raised without these figures, and that the matter is worthy of confirmatory investigation, with possible modification in the outline of the test at some later date.

J. O. PRESTON,* JUN. AM. SOC. C. E. (by letter).—The Committee, on page 1386, invites information on the subject of "proper methods of sampling highway materials." This work is usually done in the field by inspectors who may or may not have written laboratory instructions. The instructions usually convey to the inspector a notion of the need for an average sample. It has been the writer's experience that the "average" notion is much overworked. Mr. Preston.

Materials that vary between extreme limits over the work are said to average well. This is especially true with concrete materials. As a result, in many cases the variations in density of adjoining batches of concrete cause cracking, due to irregularities in the grading of the sand and stone, and the consequent widely varying coefficient of expansion. A composite sample of an aggregate from even a small area rarely indicates the actual conditions. If the average is uniform throughout the work, or if all the materials vary but slightly from the average, such a sample would be a fair specimen. In practice, however, this is seldom the case.

Experience has proved the inadequacy of leaving to field inspectors the determination of the quality of materials. For the same reason, it is inadvisable that the selection of samples from varying grades of materials, such as sand and stone, be left to the average inspector's "eye". The only safe procedure is to have trained men in the laboratory for the purpose of going into the field and collecting all samples.

It is believed that it will prove ultimate economy for State or municipal laboratories (and all other large-scale organizations), if instead of awaiting samples to be sent in for test, they develop an organization to collect them. This organization would prove of

* Rochester, N. Y.

Mr. Preston.

great value in many other ways; not only would laboratory control be more complete, but field inspectors could be instructed in the proper use of materials. (Engineers seldom have time for sufficient instruction in the field.) The slight additional cost would be cheap insurance; it would aid both the engineer and the contractor to obtain better materials and prevent their misuse or adulteration.

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Paper No. 1422

Discussion

THE DISTRIBUTION OF STRESSES IN MITERING LOCK-GATES, WITH SPECIAL REFERENCE TO THE GATES ON THE PANAMA CANAL*

By David A. Molitor, M. Am. Soc. C. E.

Mr. Molitor.

David A. Molitor,† M. Am. Soc. C. E. (by letter).—The subject of this paper is one with which the writer was concerned as early as 1893, in connection with the steel gates of the Poe Lock at Sault Ste. Marie, Mich., and several times since, until 1912, in connection with the Keokuk Canal lock.

While engaged on the Panama Canal work, in 1907-08, as designing engineer of the emergency dams, at the same time that the author was similarly engaged on the design of lock gates, this paper was practically prepared, and little of any consequence happened with which the writer was not conversant. It is surprising, therefore, that the paper should appear at this late date, especially when the method was published previously in the United States Deep Waterways' report of 1900.

It is admittedly true that this problem is fairly simple in principle, but, when the cumbersome analytic method of least work is used, it assumes enormous proportions when dealing with many redundants. The somewhat laborious computations alluded to in the paper actually occupied the author's entire time for more than a month, with the assistance of two engineers and one draftsman, working with the aid

* This discussion of the paper by Henry Goldmark, M. Am. Soc. C. E., was received too late to be printed with that paper in *Transactions*, Am. Soc. C. E., Vol. LXXXI, p. 1621.

† Detroit, Mich.

Mr. Molitor. of practically every known computing device, from the "Millionaire" multiplying machine, down to the Thacher rule.

The author's term "theory of elastic work" would make it appear that he was proposing something novel. His Equation (7) expresses Menabrea's law, or "theorem of least work", which—stated in words—means that "the redundant or indeterminate conditions reduce the actual work of deformation of a frame (or isotropic body) to a minimum". Hence there can be no doubt regarding the theorem on which the author bases his solution.

That this solution is "more complete and accurate than any previously developed" cannot be admitted at this time, though it was true at the time of publication in 1900.

The author has had ample opportunity to familiarize himself with the far more elegant and practical method of deflections, frequently brought to his attention by the writer, but apparently he could not divorce himself from his older friend "least work".

A casual reading of the paper will show that the preliminary computations, necessary for the coefficients involved in the final equations, are very laborious and had to be largely omitted. The solution of the final equations is really a minor portion of the whole work. The numerical coefficients in these equations are all large numbers, about the million mark, which the author abbreviated to five significant figures. They represent complex functions which cannot be appraised accurately by any check methods, so that duplicate computations are necessary before attempting the solution of the final equations.

The substitution of the static equilibrium conditions—Equations (1) and (2)—in the work equations also involves considerable labor.

If the effect of shearing stresses was included, such a problem would prove almost prohibitive, and yet it cannot be assumed that these are negligible quantities, by any means. The deflections of the horizontal arches due to shear alone are usually about 10% of those due to bending only; and, for the vertical stiffeners, the difference might be greater or less, depending on the style of web, which is part open and part solid.

All these difficulties are obviated by using Professor Mohr's work equations in conjunction with Professor Maxwell's law as a basis for the analysis. This leads to the far more elegant and comprehensive method of deflections, by which the preliminary computations are vastly reduced, especially when the required deflections are found by graphic methods.

The coefficients in the final equations then represent deflections which can be expressed in inches to three, possibly four, significant figures, and the final solution of these equations can be effected with an ordinary 10-in. slide-rule. Furthermore, the preliminary computations are practically done away with, because the deflections are all

found from graphic deflection diagrams, which are easily and quickly drawn, and yield results which are more accurate than the analytic, besides being readily checked. Mr. Molitor.

This is true, because a deflection diagram can be drawn for the combined effect of shear, bending, and direct stress, for a variable moment of inertia in any girder, which problem would lead to insurmountable difficulties, if attempted analytically. It is in this one respect that the author is compelled to make approximations which are wholly unnecessary. Thus, working with average cross-sections, average moments of inertia, and neglecting shear, will certainly not lead to the same high degree of accuracy as when these factors are duly considered. Also, by an appropriate choice of scales and pole distances, any desired accuracy can be attained by graphics, as, for instance, 20 000 times actual to the scale of lengths chosen.

To verify all this, the writer solved the same problem complete in 32 working hours, with the help of one assistant and a 10-in. slide-rule. This included drawing all deflection diagrams and solving the equations. It was proposed at the time to utilize this problem as a chapter in the writer's treatise* then in course of preparation, but, owing to the great number of tabulated computations and diagrams involved, it was decided later to exemplify the method on a smaller problem, thus adding to the clearness of presentation which would otherwise become obscured by a mass of figures. The complete theory and solution of this problem is illustrated on one of the 1909 steel gates of the Erie Canal, which, together with a full discussion, constitutes Chapter XIV of the above named treatise. All the fundamental equations dealing with least and virtual work, and graphic deflection polygons, influence lines, etc., are exhaustively treated in this volume. Mr. Goldmark, however, preferred the more circuitous and laborious method of solution.

Although the exact solution of this problem is possible, either by applying the method of (most) "least work" or method of deflections, it would involve as many redundant conditions, x, as there are intersections between horizontal and vertical girders, which, for the author's problem, would have necessitated the solution of $9 \times 16 = 144$ simultaneous equations. Hence, the short cut, of consolidating all material contributing to vertical stiffness into one hypothetical vertical girder, is certainly pardonable. The writer fully indorses this simplification, and adopted it for his own solution, as a practical expedient, but shows how the more exact solution may be accomplished.

As the writer's method, illustrated with a complete problem, is made available to the Profession in his before-mentioned treatise, it is not desired to inflict the burden of repetition on this Society, and hence the reader is invited to consult that treatise for full details.

* "Kinetic Theory of Engineering Structures", McGraw-Hill Book Co., 1911.

Mr. Molitor.

Regarding the solution of simultaneous equations, given in the Appendix to the paper, the writer has made several improvements which greatly reduce the numerical operations, especially the checking, in order to avoid the carrying forward of errors, and, by an appropriate arrangement of the given equations in separate tables, no operations involve more than one sheet at a time. This is also fully explained in his treatise.

The author makes this statement:

"It seems proper to put on record some of the results obtained, especially as American literature on lock-gates is very scanty. It is believed that the method of calculation used is novel, and is an advance on previous practice."

In view of the facts here presented, such a statement can scarcely be accepted, in so far as it relates to the problem of stress analysis.

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Paper No. 1423

DETENTION RESERVOIRS WITH SPILLWAY OUTLETS AS AN AGENCY IN FLOOD CONTROL*

BY H. M. CHITTENDEN, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. ARTHUR E. MORGAN, ALEX. RICE MCKIM, T. KENNARD THOMSON, ADOLPH F. MEYER, WILLIAM M. HALL, FRED. H. TIBBETTS, H. A. PETTERSON, IVAN E. HOUK, KENNETH C. GRANT, AND MORRIS KNOWLES.

1.—SYNOPSIS.

The writer's attention was particularly drawn to a possible working relationship between reservoir and spillway, as hereinafter set forth, by his study of the detention reservoir system while acting as consulting engineer to the Miami Conservancy District on certain occasions during the 30 months prior to the adoption of the Official Plan for flood protection in that district. He became convinced that applications of the detention principle were by no means exhausted in the Miami system, elaborate and well-considered as it is, but that there is a wider scope to its utility than even that notable example would indicate. The purpose of this paper is to inquire how far, and by what means, the detention principle may be made to harmonize the conflicting conditions, in reservoir development, between flood control and storage for industrial or other use. It is generally conceded that flood control of itself will not justify a very wide reservoir development, and that such justification must be sought in storage for use. It is further generally recognized that flood control and storage for use are

* Presented at the meeting of October 17th, 1917.

essentially antagonistic purposes which make impossible a reliable utilization of the same reservoir space for both. In spite of these adverse conditions, however, there is a sub-conscious feeling among engineers that two such important purposes ought to be brought into more harmonious relations. In line with this sentiment, the writer will endeavor in this paper to indicate a method by which this most desirable end can be at least partly, if not wholly, attained.

In view of the widespread interest in the question of flood control which has developed during the past few years, it is hoped that members of the Society, and others as well, who share this interest, will contribute from their experience or study to a further elucidation of the subject. A consensus of well-considered opinions, by men professionally qualified, cannot fail to be of public value.

2.—Detention Reservoirs: Definition and Terminology.

A detention reservoir is one formed by a natural or artificial engorgement in the valley of a stream whereby the volume of water which may pass for a given depth in a given time is strictly limited. The surplus which cannot thus pass in time of heavy flow accumulates in the basin above the engorgement and flows out as the inflow decreases. The function is purely automatic, human control being wholly eliminated. The effect is to prolong the period in which an excessive run-off passes the point of engorgement and to reduce proportionally the rate of discharge in the valley immediately below.

The engorgement may be produced in a variety of ways, as by the contraction of a natural chasm, by the construction of a dam with one or more conduits through the base, or by the construction of a dam with a spillway at some level above the base. In the first two cases the accumulated excess flows out entirely after the flood is past and the basin above is left empty. In the third case there will remain a body of water in the basin after the spillway has performed its alloted function. Although the first two types will be referred to frequently in this paper, only the third will be given special consideration, as it is the type which seems to be adaptable in some degree to the compromise of conflict referred to in Section 1 of this paper.

A brief reference to the terminology of the subject may properly be made at this point. The terms "detention reservoir" or "detention

basin", "retarding basin", "restraining reservoir", "impeding reservoir", have been used in official discussions of the subject. There is one defect common to all these terms, except possibly the third, and that is this—they do not suggest the end for which the system is designed, but only the means of attaining that end. Reduction of flood discharge is the great desideratum, but there is no hint of this in three of the foregoing terms. The terms "reduction" or "restriction" would be more appropriate, but there is something about these words that does not appeal to the ear. The term "restraining" carries a suggestion of both end and means, is in itself an agreeable descriptive, and perhaps fills all the requirements better than any other term that could be selected. Common use, however, is the final arbiter of all questions of this sort, and, on this basis, "detention reservoir" has the lead, notwithstanding the great prominence given to "retarding basin" by its use in the Official Plan of the Miami Conservancy District. As between "basin" and "reservoir", the writer distinctly prefers the latter, except in those cases where the basin itself and not its content is specifically referred to. As the agencies herein discussed are in the most literal sense reservoirs, "detention reservoir" is definitely adopted in this paper.

The term "dry reservoir" appeals to the writer as a most happy descriptive of that particular type of detention reservoir in which the detained excess of run-off runs out soon after the crisis of the flood, leaving the basin above the engorgement empty or dry. The original detention reservoir formed by the *Digue de Pinay* in France more than two centuries ago was of this type, as are also to be all the units of the Miami system.

3.—The Spillway: Definition and Description.

In its ordinary use the spillway is a device for conveying, over or through the crest of a dam, the surplus inflow from above in such a way as not to endanger the integrity of the dam. Its purpose is negative rather than positive. It plays no active part in the dynamic functions of the dam, as a penstock and other appurtenances do, but simply stands guard to see that the dam shall not be wrecked by any inflow in excess of that which its purpose of utility requires.

The successful operation of the spillway requires that it satisfy three vital conditions. It must have sufficient capacity to prevent overflow of the dam, because such overflow is always dangerous, and,

in the case of earthen dams, fatal unless promptly checked. The spillway must have sufficient structural resistance to withstand the tremendous strain of deep overflow. Means must also be provided at the point where the overflow strikes the level of the stream below to neutralize the energy developed in its fall without danger of undermining the dam. Inadequate satisfaction of these conditions is more common than the Engineering Profession likes to admit, and Professor Mead very properly says* that "perhaps there has been no more frequent cause of the failure of dams than inadequate spillways."

Inasmuch as the spillway requires a certain depth of overflow to be effective, and as it sometimes happens that the resulting elevation of water surface above the dam may be objectionable, the escape of water in such cases may be accelerated by the use of sluiceways, movable gates, flash-boards, and similar devices, all of which are under direct control. These serve the primary purpose of the spillway in facilitating the escape of surplus water, but lack the automatic character which is a distinctive feature of the true spillway.

The writer has defined the spillway, in the common acceptation of the term, as a safety device pure and simple. In the following discussion he will treat of it under a somewhat broader conception—one based not at all, or at least only incidentally, on the negative consideration of safety, but on positive considerations of utility. It will fulfill a vital function in the purpose of the dam as an agency of flood control, and will be a direct means of extending the application of the detention principle to cases heretofore considered of doubtful feasibility.

4.—Elements of Control.

As a preliminary to this section, it may be observed that the combined function of dam and spillway which denotes the true detention principle is universally exemplified in Nature. In almost infinite combination its elements of control may be found in actual operation. Every lake or pond which has an outlet is a case in point. Fig. 1 exhibits in graphic form the effect of Lake Kachess, Washington, in reducing the flow of the Kachess River in the great flood of November, 1906.† It will be noted that, as the run-off from the water-shed

* "Water Power Engineering," edition of 1915, p. 612.

† For the data on which this diagram is based the writer is indebted to G. L. Parker, Assoc. M. Am. Soc. C. E., District Engineer for the State of Washington, Hydrographic Branch, U. S. Geological Survey.

(inflow into the lake) begins to exceed the capacity of the outlet, the surplus accumulates in the reservoir, causing a rise in surface level and a consequent increase in outflow. This process continues until outflow balances inflow, at which point the curve of outflow, and the reservoir level, reach their maximum elevation, the curves of inflow and outflow cross each other, and the curve of storage passes through zero. The inflow curve always reaches its crest and is on the decline before it is overhauled by the rising outflow curve. The difference in time between the two crests varies greatly in different reservoirs for similar freshet conditions, and in the same reservoir for different freshet conditions. The net result, however, is in every case a temporary detention of a portion of the flood wave in the reservoir, a prolongation of the period of outflow, as compared with that of inflow, and a consequent reduction in the rate of discharge as the freshet passes through the reservoir.

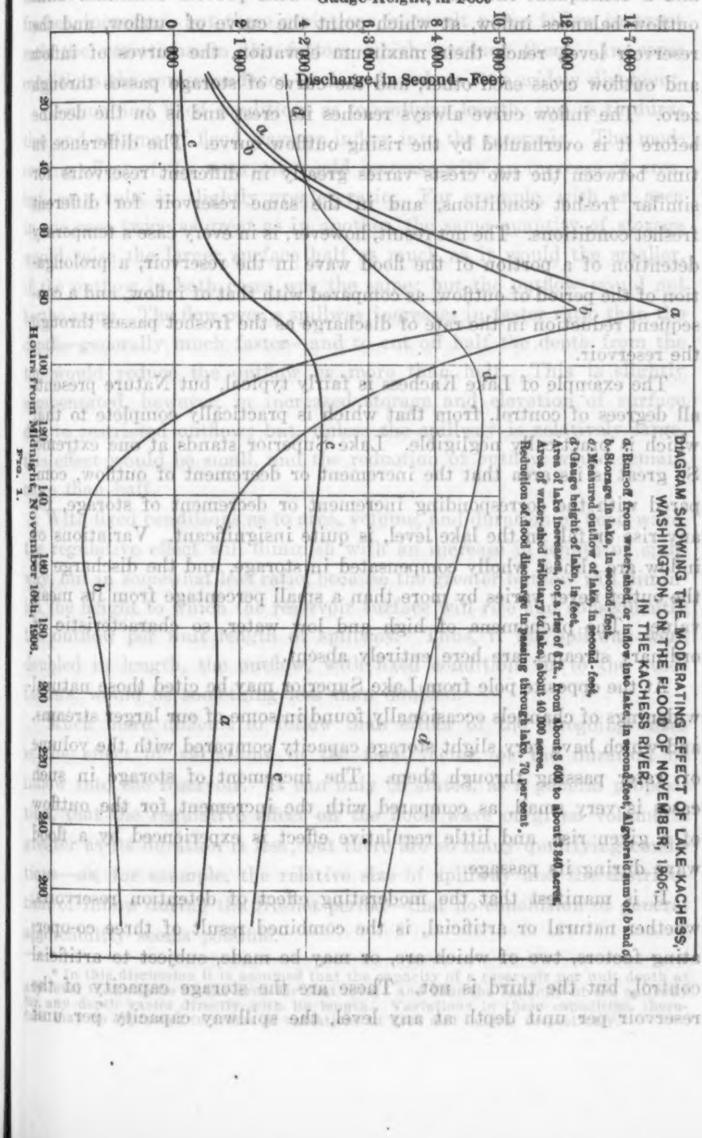

Fig. 1.

The example of Lake Kachess is fairly typical, but Nature presents all degrees of control, from that which is practically complete to that which is practically negligible. Lake Superior stands at one extreme. So great is its area that the increment or decrement of outflow, compared with the corresponding increment or decrement of storage, by a rise or fall of the lake level, is quite insignificant. Variations of inflow are almost wholly compensated in storage, and the discharge of the outlet never varies by more than a small percentage from its mean value. The phenomena of high and low water, so characteristic of ordinary streams, are here entirely absent.

As the opposite example from Lake Superior may be cited those natural widenings of channels occasionally found in some of our larger streams, and which have very slight storage capacity compared with the volume of water passing through them. The increment of storage in such cases is very small as compared with the increment for the outflow for a given rise, and little regulative effect is experienced by a flood wave during its passage.

It is manifest that the moderating effect of detention reservoirs, whether natural or artificial, is the combined result of three co-operating factors, two of which are, or may be made, subject to artificial control, but the third is not. These are the storage capacity of the reservoir per unit depth at any level, the spillway capacity per unit

(inflow into the lake) begins to exceed the capacity of the outlet, the surplus accumulates in the reservoir, causing a rise in surface level and a consequent increase in outflow. This process continues until outflow balances inflow, at which point the curve of outflow, and the reservoir level, reach their maximum elevation, the curves of inflow and outflow cross each other, and the curve of storage passes through zero. The inflow curve always reaches its crest and is on the decline before it is overhauled by the rising outflow curve. The difference in time between the two crests varies greatly in different reservoirs for similar freshet conditions, and in the same reservoir for different freshet conditions. The net result, however, is in every case a temporary detention of a portion of the flood wave in the reservoir, a prolongation of the period of outflow, as compared with that of inflow, and a consequent reduction in the rate of discharge as the freshet passes through the reservoir.

The example of Lake Kachess is fairly typical, but Nature presents all degrees of control, from that which is practically complete to that which is practically negligible. Lake Superior stands at one extreme. So great is its area that the increment or decrement of outflow, compared with the corresponding increment or decrement of storage, for any rise or fall in the lake level, is quite insignificant. Variations of inflow are almost wholly compensated in storage, and the discharge of the outlet never varies by more than a small percentage from its mean value. The phenomena of high and low water, so characteristic of ordinary streams, are here entirely absent.

At the opposite pole from Lake Superior may be cited those natural widenings of channels occasionally found in some of our larger streams, and which have very slight storage capacity compared with the volume of water passing through them. The increment of storage in such cases is very small, as compared with the increment for the outflow of a given rise, and little regulative effect is experienced by a flood wave during its passage.

It is manifest that the moderating effect of detention reservoirs, whether natural or artificial, is the combined result of three co-operating factors, two of which are, or may be made, subject to artificial control, but the third is not. These are the storage capacity of the reservoir per unit depth at any level, the spillway capacity per unit

length, and the duration of the flood wave.* In perfectly general terms, this moderating effect increases with the area of the reservoir and diminishes as the length of the spillway, or the duration of the freshet, increases; but these variations in result never follow in exact ratio the variations in the factors which produce them. In some conditions the correspondence is close, in others it is widely divergent.

Assume first fixed conditions as to spillway length, and as to duration and volume of flood wave or inflow into the reservoir. The moderating effect of the reservoir would increase with an increase of area, but, as a rule, in slightly greater ratio. For example, with an area in one case twice as great as in another, the same quantity of storage would raise the larger surface half as much as it would the smaller, if the outflow in both cases was the same; but the outflow would not be the same. The flow over a spillway increases in faster ratio than the depth—generally much faster—and to cut off half the depth from the top would reduce the outflow by more than half. This is slightly compensated, however, in increased storage and elevation of surface due to restricted outflow; but, unless the spillway is relatively large, this effect would be small, and the reduction of outflow would remain more than half.

With fixed conditions as to area, volume, and duration of flood wave, the regulative effect will diminish with an increase in length of spillway, but in somewhat less ratio, because the greater outflow will diminish the height to which the reservoir surface will rise and consequently the outflow per unit length of spillway. Thus, if the spillway were doubled in length, the outflow, with fixed conditions as to the other factors, would be something less than doubled.

Much more difficult to follow than either of the foregoing cases is the effect of variations in the time factor, or the duration of inflow into the reservoir. It can only be stated, as a general proposition, that the regulative effect on the flood wave of given volume is greater as its duration is less; but there are so many qualifying conditions—as, for example, the relative size of spillway and the distribution of inflow during the freshet period—that no conclusion of general applicability seems possible.

* In this discussion it is assumed that the capacity of a reservoir per unit depth at any level is the same as the area for that level; and that the capacity of the spillway for any depth varies directly with its length. Variations in these capacities, therefore, may be expressed in terms of variations in area and length, respectively.

Indeed, the whole problem of the inter-relation of these several factors seems to be too complicated to be expressed in any general formula which will embrace them all, and it seems destined to remain simply a problem in "particular cases"—the making of specific assumptions, and the determination of each group by itself. That is what would be done in practice anyway, but it would be a satisfaction, nevertheless, to have a general formula.

5.—Prevalence of the Reservoir Idea.

In passing from the general principles just discussed to their specific application, it may be stated at the outset that there is a deep-seated belief in the lay mind, and to a less extent in the mind of the expert, that in reservoirs is to be found the solution of the flood problem. As the writer stated some 20 years ago*: "To store the surplus water in the flood season and use it in the season of drought ought, apparently, to strike at the root of the whole difficulty. * * * Why so obvious a remedy has never yet been extensively applied," the writer at that time traced to a prohibitory disproportion between cost and resulting benefits. Although this is true, as a broad generalization, it will be more useful to the student of these questions to give some of the specific details on which the generalization rests. They may be summarized briefly as follows:

Deficiency of Sites.—This sometimes arises from an actual absence of physical sites, but more often from the lack of those which are economically feasible. Along the main valleys of the lower Ohio, Missouri, and Mississippi there are no sites whatever into which the main streams could be poured, except possibly to a small extent the overflow basins along the Mississippi. On the other hand, on almost any of the upper tributaries may be found physically practicable sites of sufficient capacity, if properly developed, to insure effective flood control. To-day, however, many of these sites are crossed by important railway systems which cannot be well re-located, or are occupied by villages and cities, rich and highly developed farms, mineral properties, etc., and, of course, almost all are traversed by public highways. It thus results that the occupancy of such sites for reservoir purposes, though physically practicable, is often economically prohibitory.

* "Reservoir Sites in Wyoming and Colorado", House Doc. 141, 55th Cong. 2d Sess., p. 32.

As a flood control measure, pure and simple, a reservoir may be only a trade-off, with the balance sheet against it. To quote again from the writer's early reservoir report (p. 46):

"Floods are only *occasional* calamities at worst. Probably on the majority of streams destructive floods do not occur, on the average, oftener than once in five years. Every reservoir built for the purposes of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. One acre permanently inundated to rescue three or four acres from inundation of a few weeks once in three or four years, and this at a great cost, could not be considered a wise proceeding, no matter how practicable it might be from engineering considerations alone."

Of course, there are other considerations of a different character than mere overflow of bottom-lands which weigh on the other side of the balance. Moreover, the "dry" reservoir quite obviates the special objection of permanent overflow.

As a rule, reservoirs are practicable only on comparatively small streams; almost never, as already pointed out, on trunk streams of great magnitude. As floods in the trunk streams are invariably the result of tributary accessions, and as the magnitude of such floods is enhanced by the synchronous arrival of tributary flood peaks, any disturbance of the rate or duration of tributary flow might have an adverse effect on the combination. Ordinarily, the reduction of a peak on a regulated tributary would more than offset such occasional adverse effect. Nevertheless, an aggravation of natural conditions is a possibility. This matter will receive further consideration later.

The effect of reduction of a flood peak on a small tributary diminishes rapidly with the distance down stream, as other tributaries come in, and amounts to little or nothing when it reaches the lower course of a large river. Even so extensive a system as that of the upper Mississippi produces no appreciable effect below the mouth of the Missouri.

Economic considerations, as already indicated, are on the whole adverse to any general development of the reservoir system for flood control alone. Industrial or other use is generally necessary to justify the cost. Flood control and industrial use, however, conflict with each other to a certain extent, as hinted in Section 1 of this paper, and as will be more fully explained in Section 6.

That the drawbacks just mentioned are not at all imaginary, but are very real and of wide application, is abundantly proved by the paucity of examples of reservoirs built for the primary purpose of flood control. There has developed in the Engineering Profession a feeling somewhat akin to pessimism on the subject, and this has been aggravated by the ill-considered advocacy of certain visionary projects which has cast doubt, if not ridicule, on the fundamental principle itself; but there is clearly a middle ground. If reservoirs alone can never solve the flood problem on all our streams—and they certainly cannot—they may probably do more than most of us in recent years have believed to be possible. Omitting those rare and extraordinary sites, relatively few in number, where dams built at slight expense, or at an expense justified by some special purpose, may control the greatest possible floods, and omitting also those situations where the dry reservoir is the most practicable type, there remains an intermediate zone of great extent in which flood control and industrial or other use are competing and more or less conflicting purposes. If something can be done to make possible the common service of these conflicting purposes in future reservoir construction, the scope of flood control by means of reservoirs may be very greatly extended. Whether or not, and to what extent, this may be done, will be our next inquiry.

6.—Conflict and Compromise.

In Paragraph 41 of a paper entitled "Flood Control"*, written in the summer of 1915, the writer has succinctly stated the nature of this conflict.

"If storms could be foreseen, both in date and intensity, this conflict could be compromised. But as they cannot be foreseen, and as experience shows that precipitation of one season may be several times greater or less than that of another, it becomes important, for storage purposes, to fill the reservoirs as soon as possible so as to be sure of a supply; while, for flood control, it is important to reserve ample space in them until the season of storms is safely past. The two purposes are thus essentially antagonistic."

The Ohio Valley Flood Board sets forth the same idea in its report of August 31st, 1916.† It says:

* Read before the International Engineering Congress in San Francisco, September 24th, 1915, and published in the *Transactions* of the Congress and reprinted as Document 2, House Committee on Flood Control, 64th Cong., 1st Sess.

† House Doc. 1792, 64th Cong., 1st Sess., p. 31, par. 56.

"If, however, a system of reservoirs should be built for power purposes, they could not, even by hypothesis, be used for the prevention of floods. Power development requires that the discharge shall be fairly uniform, and shall not fall below a fixed minimum, and also that the available head of water shall not fall below a fixed minimum. * * * Regularity of flow is so important to them that whenever water is to be had they will certainly fill up the reservoir as far as they can, so as to provide for the times of small natural discharge. This is the proper way to manage power reservoirs, and it is idle to suppose that companies which have spent money for them are going to manage them otherwise."

The State Water Problems Conference, recently authorized by the Legislature of California, refers to this matter as follows*:

"In connection with this subject careful consideration must be given to the antagonistic interests of flood control, irrigation and hydroelectric power in connection with the storing of water and its use. A reservoir for highest economic efficiency in flood control should be kept empty until actual flood; it would then be filled by the first flood, and gradually emptied after that flood had subsided in order to give storage for another flood. That same reservoir, if used for power or irrigation, would, on the contrary, be filled as soon as possible—before actual flood, if conditions permitted—and kept full, lest there should not be a subsequent flood to fill it. Such a reservoir so used could not be considered, therefore, as more than a partial factor in flood control."

The Special Committee on Floods and Flood Prevention, American Society of Civil Engineers, says in this connection†:

"If a reservoir is to be utilized to diminish floods, it is essential that it be empty when a heavy rain occurs. If to increase the low-water discharge of a river, it must be full when the rainy season ceases. To insure the first condition, it is necessary to empty the reservoir after every storm, to obtain space to store the discharge of the one following. To insure the second, it is necessary to close the outlets of the reservoir at relatively low stages, and when it is once filled, not permit the surplus water to escape until the river falls to the stage when the stored water is required for navigation."

The Official Plan of the Miami Conservancy District states the case very briefly as follows‡:

"The construction of permanent reservoirs for combined flood prevention and power purposes was found not to be feasible, because

* Report of November 25th, 1916, par. 262, small pp. 85-86.
† *Transactions*, Am. Soc. C. E., Vol. LXXXI, p. 1224.
‡ Report of Chief Engineer, February 29th, 1916, p. 84.

the same storage space cannot at the same time be used for storing water for power production, and be kept unoccupied and available for storing water in time of flood."

These expressions of opinion (and they might be extended indefinitely) indicate a well-defined conviction on the part of engineers that flood control and storage for use are, as has been repeatedly stated, "essentially antagonistic purposes." This is, indeed, perfectly true if we impose one condition, which is generally implied rather than clearly specified, that the use of the same storage space for both purposes is intended. The last citation given does clearly so state it. Under this condition no safe reliance can be placed on joint use. It might work in some cases; it will always have at least a slight favorable influence; but contingencies might at any time arise when such effect would be practicably negligible, and the reservoir be wholly ineffectual as a flood regulator. The question is not that of joint use of the same identical reservoir space, but that of having space for each purpose in addition to that required by the other. This, in fact, is the crux of the physical problem.

The possibility of securing space on the same site for both purposes is occasionally referred to in current discussions of flood control problems, but thus far without specification of details. The Official Plan of the Miami Conservancy District, for example, in the paragraph just quoted from, makes this brief reference to the subject:

"Only by creating storage space additional to that necessary for holding flood waters, can power development and flood control be provided for at the same time. In Europe such combinations have frequently worked out to advantage."

Morris Knowles, M. Am. Soc. C. E., in his Minority Report to the report of the Special Committee of this Society on Floods and Flood Prevention*, says:

"It is evident, however, that such reduced efficiency for a combination of purposes does have some value, and the maximum efficiency for each purpose can be obtained by increasing the capacity sufficiently."

Farley Gannett, Assoc. M. Am. Soc. C. E., discussing the same report, puts the whole case very clearly thus†:

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* *Transactions*, Am. Soc. C. E., Vol. LXXXI, p. 1232.

† *Ibid.*, p. 1291.

"With few exceptions, storage in the upper part of a reservoir is the cheapest, so that, by increasing the height of dams erected for other purposes to a height greater than necessary for that primary purpose, the necessary storage for flood absorption will be obtained at more reasonable cost than if a dam or reservoir is built only for flood control."

And, later, on the same page, he says:

"For these reasons the writer believes that real widespread flood control through reservoir construction and storage will be brought about by the construction of reservoirs principally for other utilization purposes, such as water supply, water power, industrial use, navigation, irrigation, and for esthetic purposes, such as park lakes, improvement, sanitation, etc."

This general idea of the combined use of a reservoir site for flood control and other purposes is characterized by the Ohio Valley Flood Board as the superposition of one reservoir upon another, and is referred to rather skeptically as follows*:

"For example (as has been sometimes proposed) a reservoir for flood prevention might be superposed on one for power development. But this would amount to having two reservoirs, since the upper part would have to be managed without reference to the lower part. Also, since the acreage overflowed increases rapidly with the height of the water, and the cost of the dam varies roughly with the square of the height, there would probably be no economy in the combination. Such superposition would therefore have to be justified by special conditions and each case be worked out on its own merit."

Likewise, the report of the Water Problems Conference of California (par. 264) refers to this subject in not very approving terms:

"Speaking generally, the same site could be used for flood control as well as for the other purposes named only by increasing the height of the dam, and having in effect two reservoirs, the top one for flood control to be filled and emptied with the coming and going of the flood, and the lower one to be filled as soon as possible and kept full as long as possible to supply water when needed. Such double construction adds materially to the cost, even if other conditions are favorable, and it must assume public control for proper use of the flood control portion of the reservoir."

The writer would bespeak for the idea thus doubtfully referred to in the last two citations a more thorough and friendly consideration than has yet been given it. May not the superposition of one reservoir

* Flood Board Report, par. 77, p. 35.

upon another contain important possibilities after all? The two reservoirs, or reservoir spaces, would be of distinct types and for distinct purposes—that below for storage, and subject to direct supervision, that above for flood control, and entirely automatic in action. The outlet of the upper reservoir would not be through conduits, as in the Miami type, but through an open spillway. Of course, in outward appearance, the two reservoirs would be one, with a single dam and a single basin above.

It will conduce to a clearer presentation of the subject if we assume what may be called an ideal example, setting forth its possible development on the foregoing lines, and then noting the qualifying conditions which must often interfere with such development, and compel its abandonment or the acceptance of something less than the highest

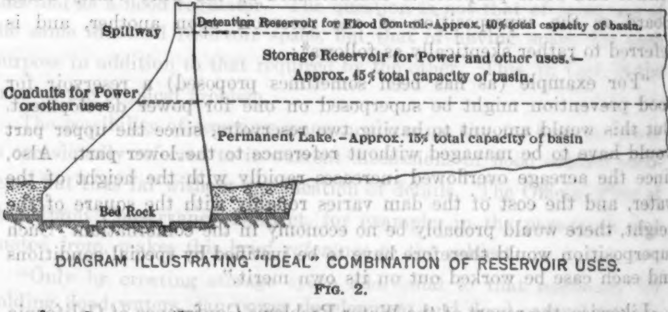

DIAGRAM ILLUSTRATING "IDEAL" COMBINATION OF RESERVOIR USES.

FIG. 2.

result. Suppose (Fig. 2) that there exists a practicable reservoir site in the valley of an important stream; that industrial and other uses in the valley can profitably utilize all the flow of the stream which could be made available; and that flood conditions in the valley are such as to justify extensive measures of relief. Let it be assumed that public authority, State or Federal, has been given the right to supervise reservoir construction, and to see that it is planned, if desirable, so as to serve all the purposes of which the development is capable.

The first consideration would be the requirements of use. It would be necessary to determine a storage capacity sufficient to equalize the flow of the stream, at least to its mean annual volume. It might indeed be very useful to do more than this, and make the excessive run-off of some years offset the deficiency of others. If it were planned to put in a power plant immediately at the base of the dam, or at least where

it could utilize the head created by that structure, it might be of advantage to make the dam permanently tight to a considerable elevation. This would insure a minimum head to be relied on at all times, and would greatly reduce the range of annual fluctuation and the resulting differences in head. With all conditions of the problem fully considered, the storage capacity would be determined, and with it the maximum level for this part of the reservoir.

The spillway would be placed at the maximum storage level just determined. The flood-control problem would then be worked out, based on the most extreme assumptions that could reasonably be made. The additional height of dam, and the spillway capacity necessary to secure the desired control, would follow, and the full dimensions of the dam would thus be determined. The portion of the reservoir below the spillway would be subject to human control; that above would not. The super-reservoir would really be of the dry type, because all that portion of the basin above the spillway level would drain out promptly after the passage of a flood, and would remain dry most of the time.

7.—Objections, Apparent and Real.

The Ohio Valley Flood Board is not strictly correct in saying* that "the upper part would have to be managed without reference to the lower part." The lower part would be managed without reference to the upper part, which would not be managed at all, but rather would manage itself—human supervision being entirely eliminated except for repairs, maintenance, and police.

As to cost, which is another matter of doubt with the Flood Board, this may be said: The capacity of a reservoir basin ordinarily increases very rapidly with its depth, or the height of the dam. In the proposed Englewood Basin of the Miami system, the following depths are required for 100 000, 200 000, 300 000, and 400 000 acre-ft. capacity: 72, 93, 107, and 119 ft., respectively. These capacities stand in almost the same ratios as the cubes of the corresponding depths, whereas, according to the Flood Board, the cost of dams increases approximately with the square of the depth. In fact, it is probably true, as Mr. Gannett has stated, that the top storage in a reservoir is generally cheapest of all.

* Report, par. 77, p. 35.

The one serious objection of the Ohio Valley Flood Board to the detention system is contained in the following extract:*

"Detention reservoirs decrease the flood height at the expense of a prolongation of the time of high water, and therefore necessarily increase the probability of coincidence of high stages from several tributary streams."

This is theoretically true, but it is so complicated with an infinitude of other factors that it is practically impossible to determine its real value. The Flood Board seems to have in mind the Ohio River more particularly. A high flood in that stream is caused by the arrival, more or less coincidentally, of the flood waves of many tributaries. In their unregulated condition, the peaks of these flood waves are sharp, their passage of short duration, and the chance of their exact coincidence correspondingly slender. Under regulation, the peaks would be flattened out more or less, and the length of the wave increased. It is quite evident that the probability of overlapping of waves would thus be increased, but this overlapping effect would probably be more than offset by the reduction in heights. At any rate it is impossible to base a conclusion on such meager data, that flood heights would be increased.

In case only one of several tributaries were regulated, it might happen that its peak would occasionally coincide with unregulated peaks, where such coincidence would not have taken place without regulation; but the chances are just as strong the other way, and here, too, the reduction in peak due to regulation would tend to offset any unusual coincidence which might occur.

The only tangible possibility of such danger which the writer can think of would be in the case of two important tributaries in the same vicinity, one of which, as shown by long records, habitually runs out enough in advance of the other to prevent their flood peaks from coinciding. If the quicker tributary alone were regulated, it might bring their peaks more closely together, and possibly result in a higher combination; but this possibility is based on so many "ifs" and improbable conditions that it is a rather unsubstantial hypothesis after all.

The utmost that can be said of these alleged risks and dangers is that they are very remote possibilities. The probabilities are all the other way. There seems to be no sufficient ground for building up, on

* Report, par. 76, p. 35.

such a basis, an adverse sentiment toward any extension of the detention principle which is otherwise practicable. Nevertheless, the Ohio Valley Flood Board feels that the matter is of sufficient importance to call for restrictive measures. It states* flatly that it "would not recommend this type [detention] unless some means of control were provided to be used when necessary, or other Ohio tributaries were similarly regulated." Later in the same paragraph it is stated: "The Board therefore does not think that reservoirs incapable of control ought to be approved by the United States unless it can be demonstrated that the danger of increased flood heights by reason of their use is quite remote."

The writer will consider somewhat in detail the two objections specified in these citations, taking that of simultaneous regulation first. It seems quite clear to him that such regulation of a vast number of streams like the tributaries of the Ohio falls outside the domain of possibility. Any such requirement would nullify all efforts at development. The system will grow little by little—one unit, not many, at a time. Better by far to take some chances and register progress than to block progress because of contingencies which are so remote as to be practically negligible.

The same caution applies as against any sweeping condemnation of the principle of automatic control. It would seem, indeed, on the face of it, that a great system of reservoirs, each subject to direct control from some central station, with a comprehensive flood-warning system giving instant data as to precipitation, run-off, gauge heights, etc., throughout the region affected, would afford more efficient control on the larger rivers than if the reservoirs were left to blind automatic operation. The matter is subject to the greatest uncertainty, however, and whatever its possible value, it would be largely offset by other considerations of direct practical importance. There would be, for instance, the immense cost of the devices necessary to control the outflow of enormous volumes of water through the dams. There would always be the risk of inefficient manipulation induced by rare use; the certainty of rapid deterioration and the likelihood of finding the mechanism not in working order when most needed; the danger of over-accumulation in the reservoir due to restriction of outflow (for that is

* Report, par. 76, p. 35.

really the form which control would take); and the probability that the interests of the valley immediately below might be subordinated to the interests, more or less hypothetical, of the trunk streams. The objection in regard to deterioration and imperfect manipulation would probably be less with spillways than with conduit outlets; but, as spillways would generally be used only where the combined purposes of flood control and storage for use are contemplated, there would be ever-present a new danger in the temptation to close the outlets in the interest of private use. The writer has set forth this danger in the following terms in his Flood Control paper (par. 43) already quoted:

"As to compromise of purpose, there will always be a peril involved in the selfishness of interests dependent upon stored water. Long periods of low water dull the public sense of danger and it becomes a contest between an aggressive and insistent private appeal and a lethargic public sense of duty. The chances are strong that private appeal will prevail; and that flood reserve space will be encroached on until a catastrophic storm comes and finds the provision made for it pre-empted. This is undoubtedly a real danger."

Taking everything into consideration, the writer believes that automatic control, divorced absolutely from the possibility of human interference, possesses advantages which far outweigh any that might result from direct control.

It is well to observe at this point that the problem of flood control by reservoirs is essentially a head-waters problem. This is true also of storage for use. Reservoir development for either or both of these purposes will never produce a material diminution of flood volumes on the lower rivers. If the Miami system could be extended to every small tributary of the Ohio, floods would come to the trunk streams just the same. The local effect would be very great, the combined effect very slight; but such effect, whatever it might be, would on the whole be favorable. It would not avoid the necessity of levees and other protective works on the lower rivers; but it would reduce somewhat the stress thereon. To that extent it would operate as a factor of safety. The magnitude of that factor, however, is not sufficient to justify the building of reservoirs on the head-waters for that purpose alone. The problem will always remain essentially a local one. The writer does not believe that either adverse or favorable possible effects on the trunk streams should have a feather's weight in determining the desirability of any local development.

Perhaps most serious of the physical obstacles to the combination of purposes herein proposed is the lack of sites, already referred to in Section 5 of this paper. On some streams there are no available sites; on some, such sites as are available have only capacity for storage for use; on some, a surplus capacity beyond that required for use will be available; and, on some, the full capacity for both purposes will be found. The conditions are infinitely variable, and, dependent on them, all degrees of possible control will be encountered. It should not follow that, because complete control cannot be had in a given case, such as can be had should not be accepted. If a reservoir can be made to care for 25, 50, 75, or any other percentage of the floods on a stream, such partial relief may be better than none at all. A reservoir with only capacity for its special use may at times afford effective control, and, even when full, will always afford some slight control; for, no matter how large the spillway, some increase in reservoir depth is necessary to bring it into play, and to that extent it acts as a detention reservoir. Every development must stand on its own bottom, and its possibilities must be exhaustively analyzed. In that way sins of omission as well as of commission may be avoided. It is of the first importance that a site once occupied be occupied to the full extent of its possibilities. If occupied by an inadequate or inferior work, this may become a permanent bar to the highest development.

8.—The Human Factor.

Probably every engineer who has had charge of responsible work has said to himself, in the bitterness of experience, that he could cheerfully wrestle with the antagonisms of Nature if he could only escape the antagonisms of Man. So, with the problem here under consideration, great as may be the physical obstacles, those which are the direct result of human agency will be found greater still. In this final section of the paper the writer will mention, rather than attempt to remove, the chief of these obstacles.

It scarcely needs be said that to carry out any such programme as outlined herein would require that jurisdiction over our streams should be vested in some public authority. As between State and Federal control, the advantage is manifestly with the latter, because it is the one authority which embraces practically all our streams from mouth to source. It alone can exercise that uniform control which is essential

to the best results. The Ohio Valley Flood Board puts the case very strongly when it states:*

"Control over our waterways for the purposes of flood prevention and protection must, in the opinion of the Board, be delegated to one central authority before any rational plan for flood control can be devised, and compliance with such a plan, or other measures for the amelioration of flood conditions properly enforced. This central authority is logically and necessarily the Federal Government."

Next comes the matter of co-operation and development. Manifestly, the additional cost required to extend an industrial scheme in any particular case to embrace also that of flood control must be borne by some form of public agency. It would be futile to insist that private interests build larger than their immediate purposes require. If more is to be done, the public must do it, and unless it be prepared for co-operation, the desired result, no matter how practicable or important, cannot be attained. Determination of the respective shares of cost to be borne by the private interest and by the public would naturally be a bone of some contention; but, as it would be a matter of expert judgment, it should not involve serious difficulty.

The *bête noire* of the whole problem will be found in making the public fund available with promptness and certainty—unless indeed an entirely new system is devised for the public financing of such projects. When a competent body of experts, designated for the purpose, has reported that a project is feasible and deserving of prompt development, and has set forth in detail the grounds on which its findings are based, there ought to be confidence enough in its recommendations to receive the full support of Congress or Legislatures. It is known, only too well, however, that hitherto this is far from having been the case, and it is to be feared that, in this broader field of co-operation, traditional methods of delay will cause energetic co-operating interests to throw up their hands in despair. Say what we will, this is one of the drawbacks of our form of government, and the public expert can never be sure of putting through a project until he has satisfied many whose interests therein do not relate to its merits at all. It is in these and similar intangible, yet very real, obstacles, arising wholly from human wrong-headedness, selfishness, and shortsightedness, that the problem which we have been discussing will encounter its greatest difficulties.

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* Report, par. 108, p. 45.

DISCUSSION

Mr. Morgan.

ARTHUR E. MORGAN,* M. AM. SOC. C. E. (by letter).—At the time of his last visit to Dayton, about a year ago, Gen. Chittenden† realized that he had not much longer to live, and stated that he considered his services to The Miami Conservancy District to be his last engineering work. He made a deep impression on the engineers and officers of the District, most of all because of his fine personal qualities.

Gen. Chittenden was first called to the service of The Miami Conservancy District when the feasibility of the retarding basin system was under discussion. He was persistently quoted at that time as being opposed to such a system, and his published writings indicated a disbelief in the value of permanent reservoirs, at least, as a generally acceptable means for flood control. Therefore, though he was personally unknown to the engineers and officials of the District, it seemed probable that he would supply a definitely critical attitude which would tend to develop any essential weakness in the proposed plan.

His first visit was coincident with that of several other engineers. Later, he returned alone, and spent more than a month in a detailed examination of the design. As he was unable to walk, his work was done almost entirely in his room, though he visited by automobile the sites of practically all the works. Mrs. Chittenden accompanied him on all his trips to care for his physical needs.

His mind was exceedingly active and restless, and he worked steadily from 8 to 12 hours a day for 7 days in the week. His demand for data from the Engineering Department seemed never to be satisfied. During the time he was with the District, the programme of the Engineering Office was interrupted to a considerable extent by his omnivorous appetite for more information. From 6 to 12 men were commonly employed in arranging and classifying data and in making calculations for him, and a considerable part of each day was spent in interviews with men in charge of the various investigations. When he had finally satisfied himself as to its soundness, he became an active partisan for the plan.

To the writer, the object of this paper seems to be, not so much to add to existing engineering knowledge, as to attempt to keep open a field of opportunity. The development of The Miami Conservancy District has established the fact that, in this instance, flood control can be secured best with retarding basins, but that their dependable use for flood control excludes their use for other purposes. The habit of following precedent is so strong that the situation in this case is

* Dayton, Ohio.

† Gen. Chittenden died on October 9th, 1917.

Mr. Morgan.

apt to be used as proof that the same constructions cannot be used in any case for more than one purpose. Apparently, the author was inspired by a desire to prevent such a premature conclusion, and to keep open the field of inquiry in other cases where similar limitations do not exist. A knowledge of the dogmatic attitudes which have been taken by engineers on the subject of flood prevention would seem to justify this effort.

1.—Combined Purpose Dams in Miami Valley.—A few points in the paper seem to merit special mention. First, it may be well to repeat that The Miami Conservancy District made no mistake in not trying to combine flood prevention with power development. Of the five dams to be built by the District, the Englewood Dam on Stillwater River offers by far the best opportunity for adding to the flood-regulating capacity of the basin storage capacity for power development. If the combination of purposes is not feasible at Englewood, it is not feasible anywhere within the District. Estimates were made of the income which would be derived if the lower third of the Englewood Basin was used for permanent storage to supply a power plant, while the upper two-thirds were used for flood control. The part of the total cost of the dam which must be charged to power development in such case is more than \$500 000; that is, such a dam would cost \$500 000 more than one with equal flood storage, but with no power storage provided. The drainage area of the Stillwater River is 650 sq. miles; the low-water flow is about 25 sec-ft.; the regulated flow with such a dam would be about 300 sec-ft.; the head would vary from 40 to 70 ft.; and, with a liberal over-all efficiency of 80%, about 1 500 continuous horsepower could be developed. In this locality, served by large steam central stations, the power would be worth not to exceed 9 mills per kw-hr. At this rate, the gross annual income would be \$88 000. To sell the power at a load factor of 40% a plant installation of 3 750 h.p. would be required, which would cost not less than \$350 000. The yearly cost of the power would include such items as the following:

Interest 5% on \$850 000	\$42 500
Depreciation, 5% on \$350 000	17 500
Taxes, 2% on \$850 000	17 000
Operating expenses and maintenance	5 000
Annual cost of power	\$82 000

The storage of water for power would permanently submerge 4 200 acres of the best farm land in Ohio, of which at least 3 500 acres will retain practically its full agricultural value under the present plan of the District. The annual rent of this land is from \$6 to \$10 per acre, making the total annual cost of the power range from \$103 000 to \$117 000, against the liberal gross income of \$88 000.

It is doubtful whether it would be a sound business undertaking to develop water power in this locality if the total annual cost of the power produced would exceed 5 mills per kw-hr. of delivered energy. At the Englewood Dam, according to the foregoing deductions, its cost would be from 10½ to 12 mills. The conditions at the other four dams of The Miami Conservancy District are still more unfavorable for power development. Mr. Morgan.

2.—Extent to Which Retarding Basins are Feasible.—Gen. Chittenden's paper might seem to infer that cases are rare in which retarding basins may be used profitably for flood control. Although the majority of flood-control problems must be solved by other methods, yet the aggregate number of cases in which retarding basin control is feasible is greater than first impressions would indicate. Over the country there are probably hundreds of cases where this method will finally be found most advantageous. The following are two typical examples that have come within the writer's experience.

The St. Francis River, in Missouri, drains a mountain water-shed of 1 500 sq. miles, and will have a maximum flood run-off of not less than 150 000 sec-ft. Through the lowlands along the Arkansas and Missouri State line the river channel has a normal capacity of less than 2 000 sec-ft. By means of an extensive system of levees, this capacity is being increased to 10 000 sec-ft., or more; but, after being improved by an extensive levee system, the channel will still have a capacity of only about one-tenth of the maximum discharge from the hills. The control of maximum floods by levees is entirely impracticable. Just above the point where the river leaves the hills is a possible dam site, where a complete control of the river can be secured at an expense which would not be a serious burden to the land affected.

The Coldwater River, in Mississippi, drains 1 000 sq. miles of hill land and then flows for many miles through the rich alluvial lands of the Yazoo Delta, the largest and most fertile body of cotton-growing land in the United States. This river, where it leaves the hills, will have a maximum flood flow probably in excess of 125 000 sec-ft., but the channel at that point has a capacity of about 900 sec-ft. Flood control of the Coldwater by channel excavation is entirely impracticable, and control by levees is greatly complicated by numerous branch streams which enter it along its course. There are two reservoir sites in the hills, the development of which would completely control maximum floods on the river. In this case, a dual purpose dam might be built with the object of storing water for rice irrigation. In these and other instances which have come to the writer's attention, where the disparity between channel capacity and flood flow is extreme, retarding basin control may prove to be the only feasible method.

3.—Spillway Versus Conduits.—The object of the paper, as stated before, seems to be to keep open the field for the combined purpose

Mr. Morgan. reservoir, and not to settle any of its details. Yet a casual reading might lead to the impression that in such reservoirs the flood openings should be spillways and not conduits. The function of the spillway as a safety factor for almost every dam should be fully recognized, but the openings for flood regulation wherever possible should be conduits and not spillways. The degree of protection below the dam is limited by the maximum flow at any time during the flood, and this maximum flow is determined by two conditions, the capacity of the openings and the capacity of the basin above the dam. The ideal control would be secured with openings which would provide a uniform rate of flow throughout the flood in the river channel below the dam, and this uniform flow should be of such volume that it would pass all the water which could not be held in the basin, below the elevation of the spillway, during the maximum possible flood. If the openings are too large, more water will be allowed to pass than is necessary, overtaxing the channel below the dam, and the reservoir will not be filled. On the other hand, if the conduits are too small to care for the maximum possible flood, the basin will be more than filled, and there will be added to the flow through the conduits a short flood due to flow over the spillway crest. The use of a spillway instead of conduits for flood regulation would have the disadvantages both of too large and too small openings. At the beginning of a flood the small flow over the spillway would be at much less than the average rate. The channel below the dam would not be filled to its capacity, and the capacity of the basin would be consumed by unnecessary storage. At the crest of the flood, the deeper flow over the spillway with its greatly increased cross-section and head would be at much more than the average rate, and the flow below the dam would be correspondingly large. So far as variation in head is concerned, the conduit and the spillway are affected alike, but, as to variation in cross-section and total discharge, the advantage is all in favor of the conduit, and this advantage is, in fact, so great as to make a spillway uneconomical for the purpose, if it can be avoided.

4.—Terminology.—The choice of the term, "retarding basin", for the works of The Miami Conservancy District was made after consideration of all the terms in use. The term, "reservoir", generally refers definitely to a place where water or other substance is held for future use. It was especially desired to avoid this inference, and so the word, "basin", was adopted instead. As between the words, retarding and detention, the latter commonly implies permanent restraint, and the former expresses exactly the function of the works of the District, to retard but not to stop the flow of flood waters. In planning a type of construction for which no designation has been generally adopted, it seemed better to choose a name which accurately describes the work than to use one which, to a greater or less degree, misstates the functions of the construction.

Without any doubt, in the numberless combinations of conditions met in flood prevention work, numerous cases will arise where dual purpose or many purpose reservoirs will be feasible and very desirable. It is well to draw public attention to this fact, and not to allow a field to be closed through prejudice or precedent. Mr. Morgan.

ALEX. RICE MCKIM,* M. AM. SOC. C. E. (by letter).—The writer believes that, in some cases, flood reduction and power storage can be adjusted by building a high dam and providing for three superimposed reservoir spaces; the lowest space to form a permanent lake for the preservation of fish life; the middle space to be used for power development, controlled by gates; and the uppermost space to be utilized for flood reduction, and controlled by a waste weir. Mr. McKim.

The waste weir must be properly proportioned. In his work, the writer could find no method in use for obtaining the dimensions of a waste weir for a reservoir. So he devised the following simple formula:

$$L = \frac{2\,A\,Q\,D - H\,P}{D\,F},$$

in which, L is the necessary length of the waste weir, in feet; A, the drainage area, in square miles; Q, the average high daily run-off, in second-feet per square mile, obtained by dividing 120 by the seventh root of A; and D, the duration in days of Q, equal to $1 + \frac{1}{70}$ of the longest flow, in miles, on the drainage area. H is the assumed height of the waste, in feet; P, the acreage of the reservoir surface at the waste crest level; and F, the maximum waste flow, per linear foot, in second-feet, for H.

T. KENNARD THOMSON,† M. AM. SOC. C. E.—The discussions on flood control by Messrs. Morgan and McKim made one glad that the redeeming feature of this Republic is, that when an experiment proves a certain procedure to be a failure, it is discarded, and an attempt is made at something better. The younger members of the Society will probably live to laugh at the crude efforts at flood control now being tried or proposed. Radically different methods will be used, and there is no reason to suppose that even the great Mississippi River cannot be controlled effectively. Mr. Thomson.

These discussions also give the speaker an opportunity to report progress on his project, "Niagara Falls Junior", for building a dam in the lower rapids of the Niagara River.

Many engineers have asked how the water, which amounts to 220 000 cu. ft. per sec., can be controlled while the dam is being built. For obvious reasons, it was advisable at first not to disclose the location of the dam, but now the speaker takes pleasure in stating that it will

* Albany, N. Y.

† New York City.

Mr. Thomson. be built on Foster's Flats, the only place in the river where there is a low shelf between the water's edge and the high bank. By using this shelf, from one-half to three-quarters of the dam can be built on dry land and carried below the bed of the river.

After this portion is built, it will be easy to divert the water from the present channel through openings in the new dam. It will then be an easy matter to complete the dam.

As the speaker was confronted with the doubts of others concerning the possibility of using the 2 000 000 h.p. of the proposed development, he wrote to the Director of the Census, Department of Commerce, Mr. Samuel W. Rogers, and asked him how much power was now used in New York State, as well as the probable rate of increase. Mr. Rogers very kindly made a most comprehensive reply to the effect that more than 3 000 000 h.p. are now used, and that the annual rate of increase is more than 300 000 h.p. From this statement, it will be seen that the normal increase for 3 years will absorb all the power which the dam can furnish on this side of the boundary line.

In addition, it may be stated that in Canada power is now transmitted successfully for 250 miles. In California, it is now transmitted 543 miles. If a circle is drawn with a radius of 500 miles, using Niagara Falls as the center, it will be found that the enclosed area will include the whole or part of twenty States and two Provinces, with 60% of the population of Canada. All this population and area would be within reach of the new dam, if enough power could be generated there. Yet, as Director Rogers' letter would indicate, there will not be enough to supply the State of New York.

Incidentally, it might be remarked that the 2 000 000 h.p. mentioned will save about 20 000 000 tons of coal per year. Surely it is time that such an enormous waste of power was stopped.

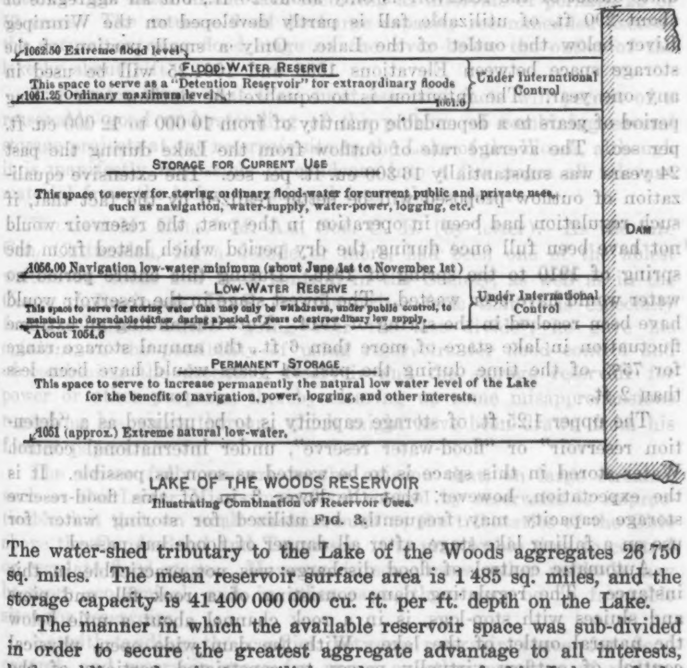
Mr. Meyer. ADOLPH F. MEYER,* M. AM. SOC. C. E. (by letter).—Gen. Chittenden's invitation to all who are interested in flood control to "contribute from their experience or study" of the subject, has prompted the writer to prepare the following notes.

In a recent report by Mr. Arthur V. White and the writer, Consulting Engineers to the International Joint Commission, in connection with the proposed regulation of the level of the Lake of the Woods and the utilization of its waters, and also in the final report of the International Joint Commission to the Governments of the United States and Canada, the combined use of a great reservoir for at least three purposes is specifically set forth. Without going into a detailed consideration of the problems dealt with in these reports, a brief statement may be made of the proposed combined use of the Lake of the Woods reservoir, for several storage purposes. Although the problem was primarily one of storage for public and private uses,

* Minneapolis, Minn.

the practicability of such use was really contingent on the successful control of extreme flood inflows into the reservoir. The utilization of a natural lake as a reservoir for artificial storage for use almost invariably involves an increase in flood stage on the lake itself, in the channel below the outlet of the lake, or both. Although foresight may prevent these increases in flood stage, there is no assurance that it will, hence provision must be made for the conditions mentioned. Mr. Meyer.

In the present instance, the "top storage", instead of being the cheapest,* is by far the most expensive. This is a condition which commonly prevails on large reservoirs that utilize natural lakes as sites. The present problem, also, is not a "headquarters problem."



LAKE OF THE WOODS RESERVOIR
Illustrating Combination of Reservoir Uses.
FIG. 3.

The water-shed tributary to the Lake of the Woods aggregates 26 750 sq. miles. The mean reservoir surface area is 1 485 sq. miles, and the storage capacity is 41 400 000 000 cu. ft. per ft. depth on the Lake.

The manner in which the available reservoir space was sub-divided in order to secure the greatest aggregate advantage to all interests, both public and private, utilizing the waters of this Lake and the shores and harbors thereof, may be better understood by reference to Fig. 3. The low-water level of the reservoir is to be held at about 5 ft. above natural extreme low water, so as to provide a minimum, during the navigation season, of 1056.0, sea-level datum; this minimum stage will also maintain permanently an increase in the available head at the outlets, reduce the cost of getting water to the power-plants at

* Pages 1485 and 1487.

Mr. Meyer. the outlets, and permit of a more satisfactory operation of saw-mills and power-plants in time of low water. Of the permanent storage below Elevation 1056, a portion is to constitute a "low-water reserve" for use only under international supervision and control, and for the purpose of maintaining the dependable outflow during a period of years of extraordinary low supply.

The storage capacity between Elevations 1056 and 1061.25, sea-level datum, is to be utilized for storing ordinary flood-water for current public and private uses, such as navigation, water-supply, water-power, logging, etc. The head available for power development at the immediate outlet of the reservoir is only about 20 ft., but an aggregate of about 290 ft. of utilizable fall is partly developed on the Winnipeg River below the outlet of the Lake. Only a small portion of the storage space between Elevations 1056 and 1061.25 will be used in any one year. The intention is to equalize the outflow over a long period of years to a dependable quantity of from 10 000 to 12 000 cu. ft. per sec. The average rate of outflow from the Lake during the past 24 years was substantially 16 300 cu. ft. per sec. The extensive equalization of outflow proposed will be better realized by the fact that, if such regulation had been in operation in the past, the reservoir would not have been full once during the dry period which lasted from the spring of 1910 to the spring of 1916. During this entire period no water would have been wasted. The lowest stage in the reservoir would have been reached in the spring of 1914. Notwithstanding an extreme fluctuation in lake stage of more than 6 ft., the annual storage range for 75% of the time during the past 24 years would have been less than 2 ft.

The upper 1.25 ft. of storage capacity is to be utilized as a "detention reservoir" or "flood-water reserve", under international control. Water stored in this space is to be wasted as soon as possible. It is the expectation, however, that the lower 3 in. of this flood-reserve storage capacity may frequently be utilized for storing water for use on a falling lake stage, after all danger of floods has passed.

Automatic control of flood discharge was not practicable in this instance. The regulating dam, consisting of a rock fill, and piers and sluices with stop-logs, is in a rock channel about a mile below the natural outlet of the lake. With the dam wide open, physical control of outflow virtually passes to constricted portions of the channel above the dam, where extensive rock excavation is required under the proposed control. Under automatic control of flood discharge, reliance is placed on increased water stages, both in the reservoir and at the spillway dam, to increase the discharge. In the present instance, substantially all the available fall in the main outlet channel is used in bringing the flood-water to the dam. In other words, increased discharge is accompanied by a drop in water level at the dam,

instead of a rise. The cost of automatic spillway control in this case would be prohibitive. Moreover, the interests on the river below the outlets of the lake make the specification of a definite, maximum, flood discharge from the reservoir highly desirable. The solution recommended is to dispose of ordinary flood inflow into the reservoir by storage, so far as practicable, below Elevation 1061.25; then to increase the discharge until the maximum permissible rate on the river below the dam is reached; and then merely to "detain" the remaining excess inflow in the space reserved for extraordinary floods. Mr. Meyer.

The extreme flood stage of 1062.5 in the reservoir is equal to extreme natural high water in the lake recurring at intervals of perhaps 25 or 50 years. The extreme flood discharge capacity recommended exceeds the natural extreme flood stage on the river below the outlet of the lake by about 1 ft.

The writer trusts that these most essential facts will permit of a reasonably good understanding of the problem of combining several storage projects at one reservoir site on the Lake of the Woods, although it is manifestly impossible to condense into a few pages all relevant material from the before-mentioned four-volume report.

WILLIAM M. HALL,* M. AM. SOC. C. E. (by letter).—As the late Gen. Chittenden, for a decade or more, had been one of the ablest writers on floods and flood control, in the Society, as well as in the Corps of Engineers, U. S. A., the writer feels that it is especially fortunate that, before "going over the last divide", he gave his opinions on the possibility of using reservoirs for flood control and of combining the detention reservoir and the storage reservoir for power or other purposes, thereby clearing up some misapprehensions regarding his views thereon, which might have been drawn from his former papers. Mr. Hall.

The writer fully concurs in the opinion that, on large streams, such as the Lower Mississippi, flood control by reservoirs is impracticable, but that, on small streams, such as the tributaries of the Ohio above the Cumberland and Tennessee, it is practicable thus to control or regulate flood stages. However, in some cases, other methods may be more economical, and, in other cases, there may be no favorable sites for reservoirs.

Notwithstanding the conflict between reservoir space for flood control, and reservoir space for power or other purposes of utility, described by the author, the writer believes that, in some cases, the two may be combined advantageously and economically if it becomes to the interest of an owner of a reservoir to use it in that way; or, such a combination may be made to the advantage of two or more owners working in harmony through a single management.

The writer wishes to suggest a thought in reference to the method of harmonizing the conflicting usage:

* Parkersburg, W. Va.

Mr. Hall. As the season of floods, in all the Eastern United States, is principally during the period from December to April; and, as the natural flow during that time will generally produce much greater power than the natural flow during the dry season, a dam or plant could be designed for the production of power exclusively by natural flow during the time of greatest floods, thereby using the upper storage space during that period for flood control and immediately thereafter filling the same storage space and using it during the dry season exclusively for power or other plant purposes. Such an arrangement would not afford perfect protection; but, at comparatively small additional cost to that for power, it could be made to reduce the flood hazard greatly. As an example, consider the flood conditions and records of the Monongahela and Allegheny Rivers, at Pittsburgh, Pa. Table 3, published by the U. S. Weather Bureau, may be used as an illustration. It is observed that, during the period of 110 years covered by this table, there were three floods in May (one of 22 ft.,

TABLE 3.—RECORD OF FLOODS IN THE OHIO RIVER AT PITTSBURGH.

Year.	Day of month.		Stage at Pittsburgh.	Year.	Day of month.		Stage at Pittsburgh.
1806	April	10	33.9	1890	March	23	24.3
1810	November	9	32.0	1890	May	24	22.0
1813	January	—	29.0	1891	January	8	23.2
1816	February	—	33.0	1891	February	18	31.3
1832	February	10	35.0	1892	January	15	23.0
1840	February	1	26.8	1893	February	8	24.0
1846	March	15	25.0	1893	February	11	22.0
1847	February	2	26.9	1894	May	22	23.2
1847	December	12	24.0	1895	January	8	25.8
1848	December	22	23.0	1896	July	26	23.0
1851	September	20	30.9	1897	February	24	29.5
1852	April	6	25.0	1898	March	24	28.9
1852	April	19	31.9	1899	March	6	22.0
1858	May	27	26.0	1900	November	27	27.7
1859	April	28	22.0	1901	April	7	22.1
1860	April	12	29.7	1901	April	21	27.5
1860	November	4	22.0	1901	December	16	25.8
1861	September	29	31.0	1902	March	1	32.4
1862	January	21	30.0	1903	February	5	24.0
1862	April	22	25.4	1903	March	1	28.9
1865	March	4	24.5	1904	January	23	30.0
1865	March	18	31.4	1904	March	4	26.9
1867	February	15	22.0	1904	March	8	23.2
1867	March	13	23.5	1905	March	22	29.0
1868	March	18	22.0	1905	December	4	23.5
1873	December	14	25.7	1907	January	20	23.3
1874	January	8	22.2	1907	March	15	35.5
1876	September	19	25.0	1907	March	20	22.4
1877	January	17	24.6	1908	February	16	30.7
1878	December	11	24.5	1908	March	20	27.3
1881	February	11	23.2	1909	February	25	22.3
1881	June	10	27.1	1909	May	1	22.2
1883	February	5	24.8	1910	January	19	22.8
1883	February	8	28.0	1910	March	1	22.0
1884	February	6	33.3	1911	January	15	23.8
1885	January	17	23.0	1911	January	31	25.2
1886	April	7	23.8	1912	March	22	28.1
1887	February	12	22.0	1913	January	9	31.3
1887	February	27	22.0	1913	January	12	26.3
1888	July	11	22.0	1913	March	28	30.4
1888	August	22	26.0	1915	February	3	28.4
1889	June	1	24.0	1915	December	19	22.6

one of 23 ft., and one of 26 ft.), one in June (24 ft.), two in July (one of 22 ft. and one of 23 ft.), one in August (26 ft.), three in September (two of 31 ft. and one of 25 ft.), none in October, and three in November (one of 22 ft., one of 28 ft., and one of 23 ft.), or thirteen floods between April and December for the entire period. It may be concluded, therefore, that had Pittsburgh had the partial protection of a series of power dams, designed with sufficient flood-storage space to hold the crest wave above a 22 to 26-ft. stage, and operated from December 1st to April 31st, it would have had, during the period, only four floods above a 26-ft. stage; or, in other words, floods which would be classed as serious disasters, an average of only one such flood each 27 years. Mr. Hall.

Although such an arrangement would not have given perfect protection, it would have prevented great property loss and damage from the other twenty-five floods above a 26-ft. stage, which it is observed occurred during that period in the five flood months of December to April.

It is believed that the execution of the project, as designed and proposed by the Flood Commission of Pittsburgh, is stopped by its great cost, and not by lack of public faith in the project *per se.* The writer thinks that a project for a series of power-flood-protection dams with the expense divided between the power companies and the public, or built at public expense and the power leased or sold, might be more easily carried to completion than one for either purpose alone.

It is believed that a study of many localities east of the Mississippi, now suffering every few years from flood disasters, may show them susceptible of similar relief. To all such localities the question of cost is the most serious one, and to many it is prohibitive. The suggested combination should help to overcome this obstacle.

FRED. H. TIBBETTS,* M. AM. SOC. C. E. (by letter).—There are many important instances in which a considerable measure of flood control may be accomplished by detention reservoirs with fixed outlets, of a somewhat different character than those emphasized by the author. Both Gen. Chittenden and Mr. Morgan, who has contributed a most interesting discussion citing a number of instances of possible flood control by detention reservoirs, could probably have mentioned other examples of the use which the writer has in mind. The following outline of the use of the Colusa Basin "rim land" as a detention reservoir will illustrate the principle involved: Mr. Tibbetts.

The Sacramento River, like most large and heavily silt-laden streams which have built up their own valley floor, now flows on a ridge flanked by overflow basins in which large bodies of land are from 5 to 25 ft. lower than the river banks. On both sides of the river the continuity of these flood basins is interrupted by ridges built up by tributary

* San Francisco, Cal.

Mr. Tibbetts. streams. In the northwest quarter of the Sacramento Valley lies Colusa Basin, one of the largest of these overflow basins, with an area of about 200 000 acres normally subject to overflow. About $5 000 000 has been spent in reclaiming this overflow land, and this work is now approaching completion. Four principal agencies are involved:

A.—Drainage canals and pumping plants, to care for surplus seepage and rain water;

B.—A continuous levee along the river frontage, to prevent overflow from river floods;

C.—A continuous back levee, to ward off, from the low lying lands, drainage naturally reaching these lands from hills, plains, and tributary streams to the westward; and

D.—A large artificial channel through the Knights Landing Ridge, which separates Colusa Basin from the next lower flood basin.

The back levee has been located so that it will exclude flood waters from the lower lands in the basin, but, during heavy storms, will allow temporary flooding of a considerable area of high land outside of the levee system. This area of excluded land, locally known as "rim land", thus acts as a detention reservoir, very greatly reducing the quantity of water which must be handled by the artificial outlet channel through the Knights Landing Ridge. The excluded land is so high that under normal conditions it drains off in a few days. It is used for farming purposes, a considerable portion having been devoted last year to the culture of irrigated rice.

The discharge through the outlet of this "rim land" or detention reservoir is fixed by the cross-section of the Knights Landing Cut itself, and the action, apparently, is closely analogous to that which will take place in the detention reservoirs of the Miami Conservancy District. The total area of water-shed tributary to the Knights Landing Cut is about 1 600 sq. miles, about one-half of which is comparatively level plain contributing but little run-off. The remainder consists of comparatively bare hills and mountains reaching a maximum elevation of about 2 000 ft.

Prior to reaching the "rim land", the run-off from about 80% of the tributary area spreads out over "Upper Colusa Basin", which in the main is unleveed, and which in itself acts naturally as a temporary detention reservoir.

The average estimated annual run-off is about 230 000 acre-ft., the maximum being 705 000 acre-ft. The back levee corresponds to the dams of the detention reservoirs of the Miami Conservancy District, and is comparable in size, the main levee line being about 25 miles in length, with a maximum height of about 26 ft., and containing in all about 11 000 000 cu. yd.

The total storage capacity of the "rim land" at the elevation of the crest of the back levee is 464 000 acre-ft., or about twice the average annual run-off. Mr. Tibbetts.

At the projected freeboard on the levees of 8 ft., the rim land will have a storage capacity of about 170 000 acre-ft., equal to about three-fourths of the average annual run-off, and about one-fourth of the maximum annual run-off. A careful analysis of flood conditions as they would have been if controlled by the present project in the 1911 flood, which is the greatest recorded on these water-sheds in the 30 years that records have been kept, shows that the rim land detention would have reduced the discharge which the Knights Landing Cut must handle from 26 000 to 17 300 sec-ft., or 8 700 sec-ft., or approximately one-half of the remaining required capacity. Actual measurements of the smaller floods of 1916, after the Knights Landing Cut had been put into operation, showed that the maximum discharge in the Knights Landing Cut was only 47% of the maximum discharge into the detention reservoir of the rim land.

The location of the back levee line in such a way as to exclude a portion of the overflow basin has permitted a great reduction in the size and cost of the artificial outlet channel, without preventing the use of the excluded land, although, of course, it impairs its value, because it prevents it from being settled, although not from being farmed. There are a number of other locations in the Sacramento Valley and elsewhere in California, where similar results can be obtained from the utilization of the high portions of the flood basins as detention reservoirs, and the writer does not doubt that there are other important instances where similar treatment is possible.

Comparative Costs.—The Miami Conservancy District is apparently placing its chief reliance for flood control on detention reservoirs, but its plans also include extensive channel improvement and levee work. The comparative costs of a given measure of flood control, where the two methods are available, is unfavorable to channel improvement and levee construction, because of the general lack of adequate equipment for work of this class in the Mississippi Valley, as compared with that in use in California.

On a recent cursory inspection of the proposed channel improvement at Dayton, the writer was somewhat surprised to be told that local contractors believed that gravel could not be handled successfully with a suction dredge "because 200 000 or 300 000 cu. yd. would wear out the pipes". The Sacramento River West Side Levee District is using a 20-in. suction dredge for levee construction, and, with this dredge, has already placed 1 830 000 cu. yd. of sand and gravel, at an average cost of 10.7 cents per cu. yd. This cost is abnormally high, because much of the work was done in short and scattered sections.

Mr. Tibbetts.

In one continuous section of about 5 miles of levee, 1 062 000 cu. yd. were placed, at an average cost of 8¼ cents per cu. yd., these figures including all overhead expenses, as well as operation, interest, depreciation, and maintenance. The material varies greatly, much of it being gravel suitable for concrete. A mechanical analysis of one section from which material is being borrowed for heavy concrete structures, shows that from 62 to 70% is gravel from ¼ in. to 1¾ in. in size. It seems to the writer that most of this material is harder and sharper than that in the river at Dayton. No new pipe has been bought since the work began, two years ago, and none has worn out. There does not seem to be any good reason for gravel wearing the pipes more rapidly than sand, except perhaps at bends. Sand is considered a better material for levee construction than clay, because of its immunity from the attacks of burrowing animals, and the fact that it does not shrink or crack.

Apparently, in the Mississippi Valley, no use has been made of the large clam-shell dredges which are the chief reliance for levee construction on the Pacific Coast. The largest of these machines have buckets with a capacity of 6½ cu. yd., and booms up to 240 ft. in length; they can handle all classes of material with ease, placing it from 300 to 400 ft. from the point where it is excavated. The writer has seen the surprising statement that "a canal with a base width of 123 ft. is practically the limit for economic construction with the floating dredge equipment now built", this comment referring to dipper-dredges.

In completing the Knights Landing Cut previously referred to, floating clam-shell dredges excavated a cross-section 6 000 sq. ft. in area, and with a bottom width of from 400 to 500 ft., without rehandling any material. By rehandling half the material, it is possible to excavate a canal of approximately double this width.

It may be because of the lack of adequate equipment that levee standards, generally, in the Mississippi Valley, are quite inferior to those in California. The Sacramento River West Side Levee District, for example, protecting the river frontage of Colusa Basin, has adopted a general standard showing a 20-ft. top width, an 8-ft. freeboard, and a volume exceeding that proposed for Dayton, and in general for work along the Mississippi River, by about 40 per cent. The writer recently saw some equipment which had been used for levee construction at Dayton, consisting of a bucket operating on a cable line suspended across the river from crude wooden towers, the whole apparatus appearing about as obsolete as a "dinosaur". He was told that the contractor had been getting more than 20 cents per cu. yd., and had "gone broke".

The channel improvement and levee construction work of the Sacramento River West Side Levee District, done by large clam-shell

and suction dredges, has cost about 40% of the foregoing figure at Dayton, though conditions are hardly comparable. Mr. Tibbetts.

It may also be possible that the cost of flood control, by levees and channel improvement as compared with detention reservoirs, in the Miami Conservancy District, is unfavorably affected by the lack of some features which are more or less standardized for large dams, such, for example, as protection on exposed slopes. Experience on the Pacific Coast at least has indicated that thick earthen embankments can be completely destroyed by wave wash in a short time, possibly within the limits of the maximum time of exposure at an essentially fixed elevation which may occur in some of the Miami detention reservoirs. The writer does not intend this in any way as a criticism of the very thorough and carefully prepared plans for the Miami work, but rather intends to indicate that, with conditions as they are on the Pacific slope, the cost of a given measure of flood control by detention reservoirs and costly dams, as compared with the same measure of control by channel improvement and levees, may be relatively greater than for the Miami Conservancy District.

H. A. PETTERSON,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper, written by so eminent a man as the late Gen. Chittenden, should help to remove some of the bias which, to the writer's mind, exists against the use of reservoirs in flood control projects. Mr. Petterson.

As one example of this biased mental attitude, from which even the author was not free twenty years ago, the following quotation† is offered:

"Floods are only *occasional* calamities at worst. Probably on the majority of streams destructive floods do not occur, on the average, oftener than once in five years. Every reservoir built for the purpose of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. One acre permanently inundated to rescue three or four acres from inundation of a few weeks once in three or four years, and this at a great cost, could not be considered a wise proceeding, no matter how practicable it might be from engineering considerations alone."

On the supposition stated, that destructive floods do not occur oftener than once in 5 years, on an average, why is it reasonable to assume that the reservoir basin will be in a condition of "permanent" overflow? As a matter of fact, most of the area of the basin would be in a dry condition oftener than lands between levees, or lands occupied by a by-pass, or auxiliary channel. For, in the latter two cases, all the land would be submerged by every flood, but only a part of the

* Tientsin, China.

† From "Reservoir Sites in Wyoming and Colorado", p. 46.

Mr. Petterson. total reservoir would be submerged by the average flood. It should be borne in mind that the reservoir capacity is designed for a flood of exceptional magnitude, which will not occur oftener, on an average, than once in 50 years, or longer, so that the basin will be completely filled only at these rare intervals of time. If the land is suitable for agricultural purposes, the greater part of it could be used for such purposes more frequently than (1) lowlands overflowed at every high water, (2) lands between the river and levee, (3) lands occupied by a by-pass or auxiliary channel.

To illustrate graphically the great probable yearly variation in water content of a flood prevention reservoir, Fig. 4 is presented. It is based on data contained in United States Water Supply Paper No. 334. Table 2 is based on the same data plotted on "probability paper" after the method proposed by Allen Hazen, M. Am. Soc. C. E.* This table shows that the reservoir would be full to capacity on an average of only once in 100 years; it would be filled to four-tenths of its capacity ten times in 100 years, or once in 10 years, on an average; and for nearly 50 years out of every 100 it would be empty throughout the whole year. It should be borne in mind that, even during flood years, the basin would be completely empty during part of the year.

TABLE 2.—YEARLY VARIATION IN STORAGE REQUIREMENT, FROM DATA OF FIG. 4. RESERVOIR FOR FLOOD PREVENTION.

Reservoir capacity as percentage of $V_m = p\ V_m$.	Percentage of years that $p\ V_m$ will be equalled or exceeded.	Percentage of years when capacity less than $p\ V_m$ will suffice.
100 V_m	1	99
58.5 V_m	5	95
40.2 V_m	10	90
21.8 V_m	20	80
11.5 V_m	30	70
8.0 V_m	40	60
2.8 V_m	50	50
0 V_m	55	45

V_m = maximum capacity for which reservoir is designed.
p = a percentage.
Values of p are tabulated in the first column.

Another instance of mental bias toward reservoirs is exhibited, to the writer's mind, in the statement that the same reservoir cannot be used for both flood prevention and storage for use, as the two pur-

* "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply", *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

Mr. Petterson.

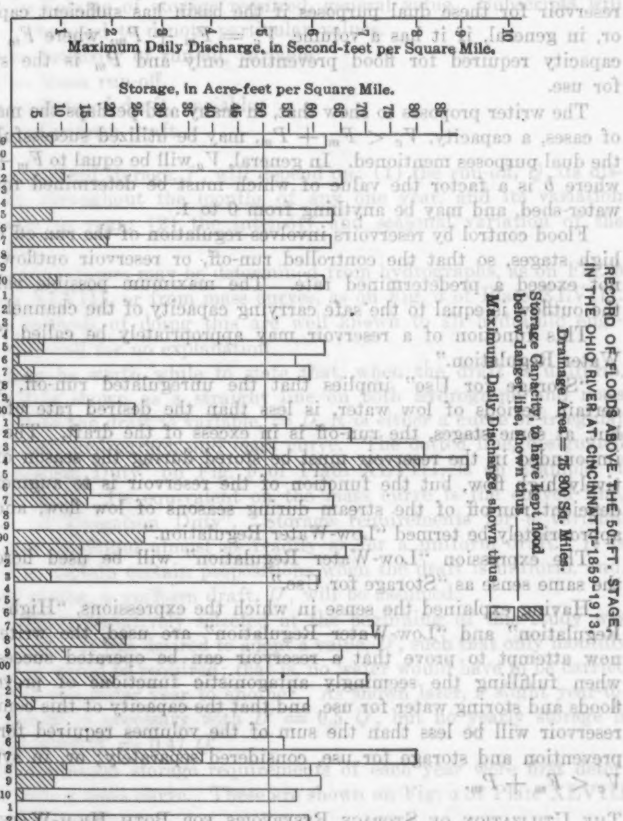


Fig. 4.

Mr. Petterson. poses are antagonistic. The author has cited a long list of statements bearing on this subject in that part of his paper headed "Conflict and Compromise". He shows that it is possible, of course, to operate a reservoir for these dual purposes if the basin has sufficient capacity; or, in general, if it has a volume, $V_c = F_m + P_m$, where F_m is the capacity required for flood prevention only and P_m is the storage for use.

The writer proposes to show that, in many and perhaps the majority of cases, a capacity, $V_c < F_m + P_m$, may be utilized successfully for the dual purposes mentioned. In general, V_c will be equal to $F_m + bP_m$, where b is a factor the value of which must be determined for each water-shed, and may be anything from 0 to 1.

Flood control by reservoirs involves regulation of the run-off during high stages, so that the controlled run-off, or reservoir outflow, shall not exceed a predetermined rate. The maximum possible value of the outflow is equal to the safe carrying capacity of the channel.

This function of a reservoir may appropriately be called "High-Water Regulation."

"Storage for Use" implies that the unregulated run-off, during certain periods of low water, is less than the desired rate of draft, but, at some stages, the run-off is in excess of the draft. The water impounded in the reservoir must be stored during the season of relatively high flow, but the function of the reservoir is to augment the deficient run-off of the stream during seasons of low flow, and may appropriately be termed "Low-Water Regulation."

The expression "Low-Water Regulation" will be used herein in the same sense as "Storage for Use."

Having explained the sense in which the expressions, "High-Water Regulation" and "Low-Water Regulation" are used, the writer will now attempt to prove that a reservoir can be operated successfully when fulfilling the seemingly antagonistic functions of preventing floods and storing water for use, and that the capacity of this composite reservoir will be less than the sum of the volumes required for flood prevention and storage for use, considered separately; or, in symbols, $V_c < F_m + P_m$.

The Utilization of Storage Reservoirs for Both High-Water and Low-Water Regulation.

To simplify the discussion, it will be taken up under four heads, namely:

A.—Reservoir Capacity for Low-Water Regulation Only.

B.—Reservoir Capacity for High-Water Regulation Only.

C.—Reservoir Capacity for the Combined Function of Both High-Water and Low-Water Regulation.

D.—The Operation of a Reservoir Having This Dual Function.

A.—Reservoir Capacity for Low-Water Regulation Only. Mr. Petterson.

The following symbols will be used:

P = Volume of storage required, general value. Subscripts will be used to denote particular values.
Q = Intensity of run-off, variable.
Q' = Mean run-off.
D = Rate of draft, variable.
D' = Uniform draft.

The required storage, P, will depend on: (1) the run-off, Q, its distribution throughout the months of any one year, and its variation from year to year; (2) the intensity and seasonal variation of the draft, D.

Storage volumes may be determined from hydrographs, as on Fig. *b* of Plate XLVIII, or from mass curves, as on Fig. *a* of Plate XLVIII. As the methods of doing this are well known to all hydraulic engineers, they call for no explanation.

It may be worth while to state that, when the draft is uniform, $= D'$, it is shown as a straight line on both hydrograph and mass curve; when the draft is variable, $= D$, it is either a curve or irregular line on both hydrograph and mass curve. The dotted line, designated as "Irrigation Duty" on Fig. *b* of Plate XLVIII, is an example of variable draft. Its equivalent on the mass curve is the curved line, "Vector of Irrigation Duty". Storage requirements for a variable draft may be found almost as readily as for a uniform draft. It is simpler to explain certain propositions when the draft is uniform, and, for that reason, a uniform draft, D', will be assumed.

The draft tentatively selected at the beginning of this study was $D' = 0.5\ Q'$. It was desired to adopt a value, D', such that only monthly regulation would be required; that is, no water would have to be carried over from one water year to another. As shown later, a slight year-to-year storage is necessary with $D' = 0.5\ Q'$, but no yearly storage is required when $S' = 0.47\ Q'$.

The maximum storage requirements of each year were first determined from a mass curve. These are shown on Fig. *a* of Plate XLVIII as P_3, P_4, P_5, etc.

Their volumes, in thousands of acre-feet, are assembled in Table 3 in order of magnitude.

How are we to know that the largest of these values will suffice for the reservoir? Perhaps if there had been 10 or 20 additional years of records, a capacity considerably greater might have been found necessary. The paper by Mr. Allen Hazen, entitled "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply,"* furnishes the

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

Mr. Petterson.

TABLE 3.—LOW-WATER REGULATION STORAGE, IN ORDER OF MAGNITUDE.
$D' = 0.5\ Q'$

Year.	P, in thousands of acre-feet.
1906	300
1909	300
1912	450
1908	780
1907	900
1911	1 050
1904	1 200
1910	1 350
1915	1 600
1903	2 900
Σ	10 830
Mean.	1 083

TABLE 4.—YEARLY RUN-OFF, IN THOUSANDS OF ACRE-FEET.
(Water Supply Paper No. 300, pp. 449-451.)

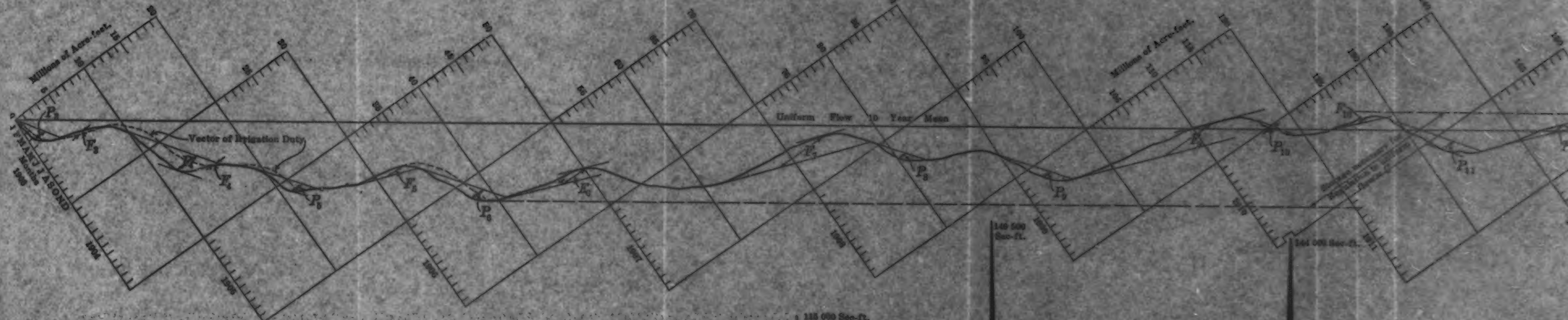
Year.	Run-off.	Variation from the mean = V.
1902	7 956	− 8 644
1903	11 364	− 5 236
1904	10 117	− 6 483
1905	19 698	+ 3 098
1906	19 472	+ 2 872
1907	25 510	+ 8 910
1908	13 707	− 2 823
1909	25 996	+ 9 396
1910	14 383	− 2 217
1911	17 824	+ 1 224
Total.....	166 027	

Mean = 16 600 000 acre-ft.

$$Q' = \frac{16\ 600\ 000}{723} = 23\ 000 \text{ sec-ft.}$$

answer to these questions, and is by far the best discussion of this subject that has yet appeared in engineering literature. Mr. Hazen applies the theory of probabilities in a very ingenious manner. A thorough understanding of that paper is essential to a clear comprehension of parts of the writer's discussion. Figs. 5 and 6 are reproduced from Figs. 26 and 39, of Mr. Hazen's paper. Fig. 5 gives the volume required for regulation during any one water year; Fig. 6 gives the annual storage, P_y, which must be carried over from year to year. The required reservoir capacity, P_m, will be the sum of the two volumes found from Figs. 5 and 6.

FIG. a.-MASS CURVE OF COLORADO RIVER, AT YUMA. 10-YEAR PERIOD.



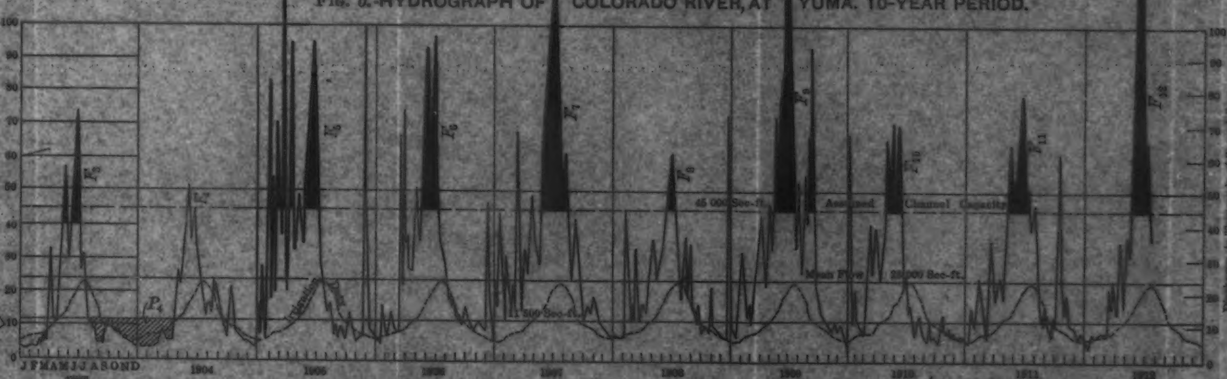
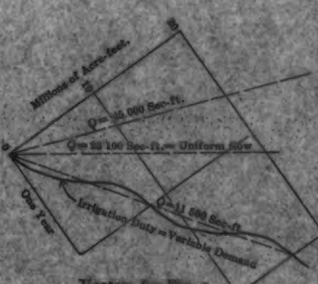


FIG. b.-HYDROGRAPH OF COLORADO RIVER, AT YUMA. 10-YEAR PERIOD.



Vectors for Fig. a

Storage for flood regulation —

Storage for regulation to 11 50

Relative storage capacities

FIG. *a*.-MASS CURVE OF COLORADO RIVER, AT YUMA. 10-YEAR PERIOD.

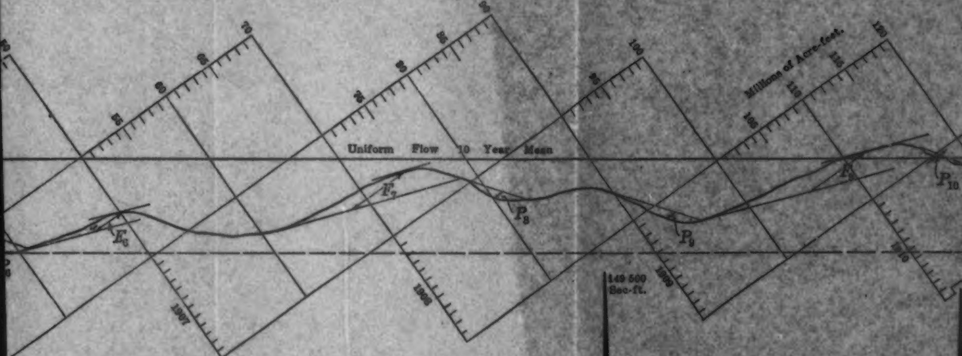


FIG. *b*.-HYDROGRAPH OF COLORADO RIVER, AT YUMA. 10-YEAR PERIOD.

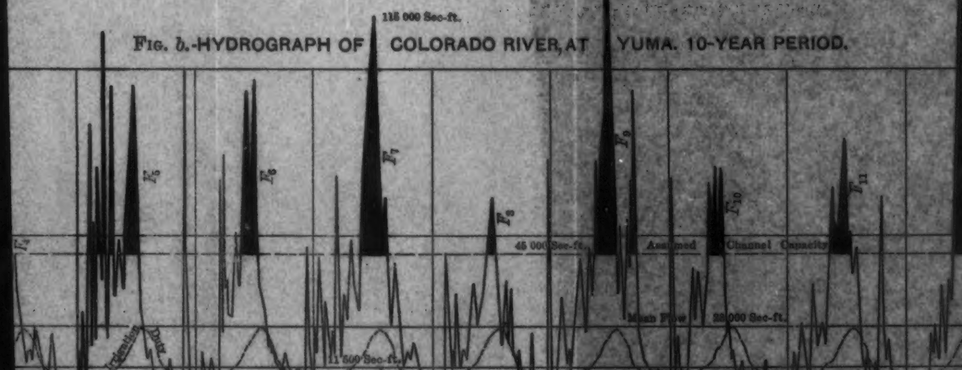
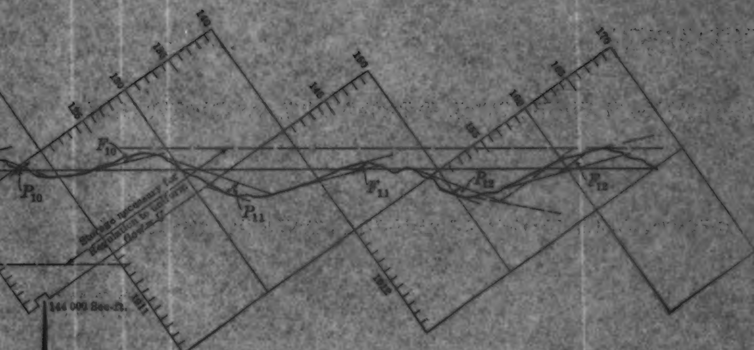


PLATE XLVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXII, No. 1423.
PETTERSON ON
RESERVOIRS FOR FLOOD CONTROL.



Storage for flood regulation = F_3, F_4, F_5 etc. Maximum = F_7 and F_9

Storage for regulation to 11 500 Sec-ft. = P_3, P_4, etc. Maximum = P_4

" " " " 23 100 Sec-ft., (uniform flow) = U

Relative storage capacities $\left\{ \begin{array}{l} U \\ F_7 = F_9 \\ P_4 \end{array} \right.$

Mr. Petterson.

The annual storage, P_y, can be determined directly from Fig. 6 for any stream for which the following terms are known:

Q' = mean rate of run-off;

D' = rate of draft;

c = coefficient of variation, $= \dfrac{V_s}{Q'}$;

V_s = standard variation $= \sqrt{\dfrac{\Sigma V^2}{n}}$, where V is the difference between the volume of mean yearly run-off and the run-off for any one year;

n = the number of years of record.

Application to the Stream Flow of Plate XLVIII.—$C = \dfrac{V_s}{Q'} = 0.353$, for the data of Table 4, where the values of V are given in the third column. $n = 10$.

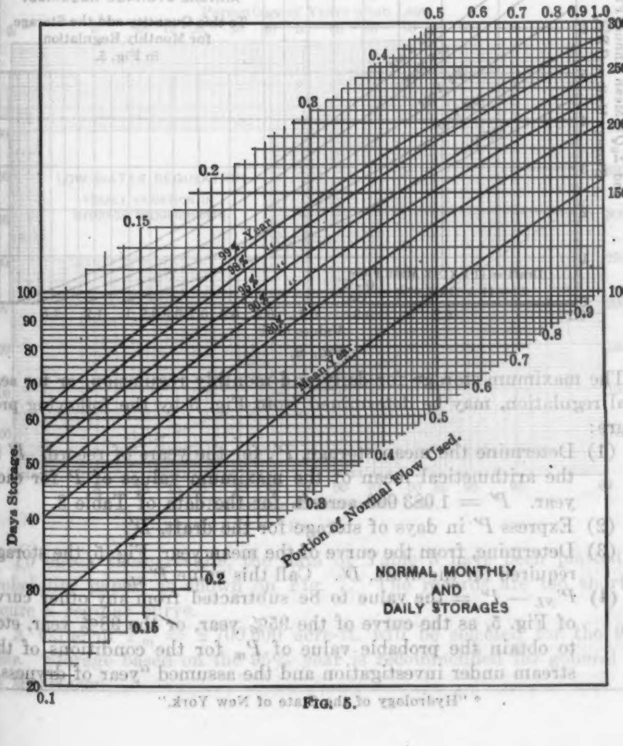


FIG. 5.

Mr. Petterson.

The abscissas of Fig. 6 $= k = \dfrac{Q' \; D'}{c \; Q'} = \dfrac{Q' \, (1 - 0.5)}{Q' \times 0.353} = 1.42$, nearly.

The annual storage requirement for the 95% year is seen to be almost negligible for $k = 1.42$, and would be zero for $k = 1.5$.

The corresponding value of D' is $0.47 \; Q'$ (for $k = 1.5$).

This would be the maximum rate of draft to use for low-water regulation for the minimum cost of the reservoir per second-foot of draft. Storage carried over from year to year is not considered economically advisable by many engineers, except for uses where the water has a very high value. The late George W. Rafter, M. Am. Soc. C. E., advises* against the adoption of rates of draft so high that yearly storage is imperative.

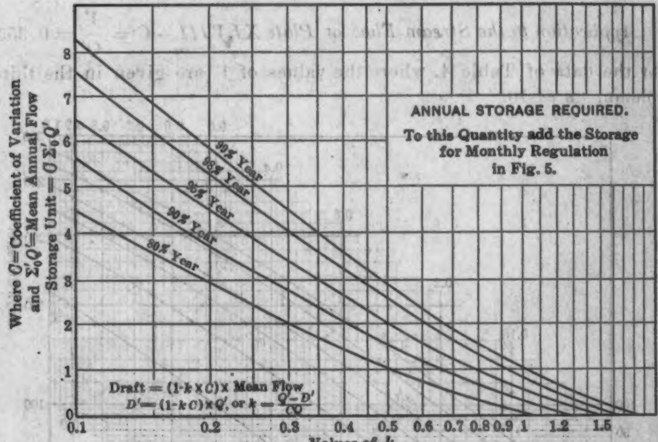


Fig. 6.

The maximum storage for daily and monthly regulation, or for seasonal regulation, may be determined from Fig. 5 by the following procedure:

(1) Determine the mean storage, P', for the years of record. P' is the arithmetical mean of the maximum values of P for each year. $P' = 1\,083\,000$ acre-ft. for the data of Table 3.

(2) Express P' in days of storage for the draft, D'.

(3) Determine, from the curve of the mean year, Fig. 5, the storage required for the draft, D'. Call this value P'_{NL}.

(4) $P'_{NL} - P'$ = the value to be subtracted from any other curve of Fig. 5, as the curve of the 95% year, or the 98% year, etc., to obtain the probable value of P_m for the conditions of the stream under investigation and the assumed "year of dryness."

* "Hydrology of the State of New York."

From Tables 3 and 4, the following values are obtained: Mr. Petterson.

$\Sigma Q' = 16\,600\,000$ acre-ft. per year.

or $Q' = 23\,000$ sec-ft.

$D' = 0.5\,Q' = 11\,500$ sec-ft. $= 22\,800$ acre-ft. per day.

(1) $P' = 1\,083\,000$ acre-ft.

(2) $P' = 47.5$ days storage at rate D'.

From Fig. 5:

(3) $P'_{NL} = 100$ days storage at rate $D' = 0.5Q'$.

(4) $P'_{NL} - P' = 52.5$.

P'_{NL} (95% year) $= 168$ days.

P'_{NL} (98% year) $= 185$ "

Probable storage for stream of Plate XLVIII:

P_m (95% year) $= (168 - 52.5)$ days $= 2\,615\,000$ acre-ft.

P_m (98% year) $= (185 - 52.5)$ " $= 3\,022\,000$ acre-ft.

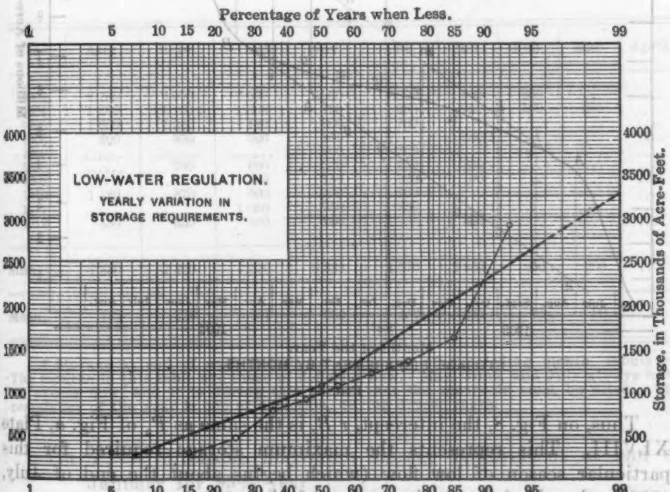


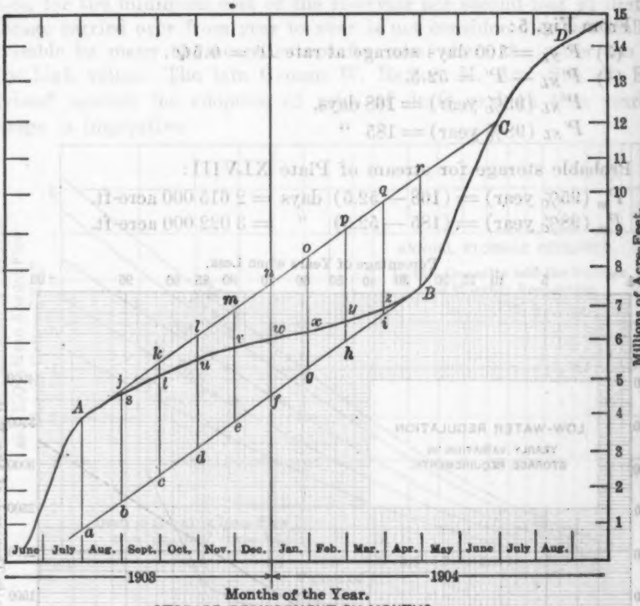
Fig. 7.

To check these results, the data of Table 3 have been platted on probability paper, as shown on Fig. 7. The records are too short to secure a regular curve.

A value of $P_m = 2\,700\,000$ acre-ft. will be selected for the 95% year. Storage based on the 95% year is recommended for general use by Mr. Hazen.

Mr. Petterson.

This storage, P_m, is to be used to augment the low-water flow, during periods of deficiency, so that, throughout every year,* a rate of draft, D', may be used safely. Now, it is self-evident that the reservoir should be full at the beginning of the season of low water, but, as water is drawn upon for use, the reservoir becomes gradually depleted, and at the end of the worst season, it will be entirely empty.



Months of the Year.
STORAGE REQUIREMENT BY MONTHS.

FIG. 8.

Thus, on Fig. 8, the intercept, $r\ B$, is the same as P_4 of Fig. a, Plate XLVIII. This represents the maximum storage required for this particular season of low flow (which begins about the end of July, 1903, and ends about the beginning of May, 1904).

The capacity, $r\ B$, is required at the beginning of the season of low flow; not at its termination. Thus, a volume, $A\ a = r\ B$, must be impounded by the end of July, 1903. The storage volumes necessary at the ends of other months are given in Table 5.

This method was applied to all the 10 years of record, with results as assembled in Table 6.

* Storage computed for the 95% year signifies that it will be of ample capacity in 95 out of every 100 years; but, during 5 years in a century, a greater volume will be needed.

Mr. Petterson.

TABLE 5.

	End of:									
	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.
Storage required..............	*a A*	*b s*	*c t*	*d u*	*e v*	*f w*	*g x*	*h y*	*i z*	0
Reservoir depletion..........	0	*j s*	*k t*	*l u*	*m v*	*n w*	*o x*	*p y*	*q z*	*r B*

TABLE 6.—LOW-WATER REGULATION.

Required storage,* in thousands of acre-feet, to ensure a sufficient supply during the remainder of the season of low flow. For a draft, $D' = \frac{Q'}{2}$.

Year.	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	April.
1903	2 700	2 500	2 300	1 900	1 500	800	350	....	....
1904	1 200	1 100	1 000	700	250	950	500	200	....
1905	1 600	1 360	1 120	900	700	....	....	....	....
1906	200	300	300	150	......	300	....	....	....
1907	......	......	......	900	700	....	....	....	....
1908	......	780	600	330	250	350	....	....	....
1909	......	......	300	150	220	100	....	....	....
1910	1 180	870	600	400	100	....	....	....	....
1911	......	......	1 050	950	850	....	....	....	....
1912	450	350	300	250	......	350	....	....	....
Sum.....	7 330	7 260	7 570	6 630	4 570	2 850	850	200	....
Mean....	733	726	757	663	457	285	85	20	....

* Storage at end of each month, or beginning of succeeding month, tabulated. The months in this table are those of deficient flow. May, June, and July always have an excess flow, and it is during this period that the reservoir must be filled. The remainder of the year is one of depletion, during dryest years.

Let P_1 = Probable maximum storage requirement at end of January for 95% year;

P_2 = Probable maximum storage requirement at end of February for 95% year;

P_{12} = Probable maximum storage requirement at end of December for 95% year.

The method of probabilities could be applied to the data compiled, or in Table 6, and the probable maximum storage requirements, P_1, P_2, etc., P_{12}, determined.

Mr. Petterson.

For the stream flow of Plate XLVIII, a better method is as follows:

(1) May, June, and July are the months of excess flow, when the reservoir will have to be filled.

(2) The other months of the water years are those of deficient flow, when the water impounded in the reservoir must be drawn upon. The season of deficient flow may be said to begin at the end of July and terminate at the end of April or the beginning of May.

(3) Hence, determine from the run-off records, by applying the theory of probabilities, the probable minimum run-off in the 95% year, for the following periods:

(*a*) April $= \Sigma\, Q_1$.
(*b*) March and April $= \Sigma\, Q_2$.
(*c*) February, March, and April $= \Sigma\, Q_3$.
(*d*) January, February, March, and April $= \Sigma\, Q_4$, etc., etc., etc.
(*i*) August, September, October, November, December, January, February, March, and April $= \Sigma\, Q_9$.

(4) Let the total draft for each of these periods, in the order listed under (3), be $\Sigma\, D_1$, $\Sigma\, D_2$, $\Sigma\, D_3$, etc.

Then the required storage at the end of each month is as shown in Table 7.

TABLE 7.

End of:	Storage, P.	Value of P.
July	P_7	$\Sigma\, D_9 - \Sigma\, Q_9$
August	P_8	$\Sigma\, D_8 - \Sigma\, Q_8$
September	P_9	$\Sigma\, D_7 - \Sigma\, Q_7$
October	P_{10}	$\Sigma\, D_6 - \Sigma\, Q_6$
November	P_{11}	$\Sigma\, D_5 - \Sigma\, Q_5$
December	P_{12}	$\Sigma\, D_4 - \Sigma\, Q_4$
January	P_1	$\Sigma\, D_3 - \Sigma\, Q_3$
February	P_2	$\Sigma\, D_2 - \Sigma\, Q_2$
March	P_3	$\Sigma\, D_1 - \Sigma\, Q_1$
April	P_4	0*

* No storage is required by the end of April, or the first of May, because the run-off in May is always in excess of the draft, D'.

This procedure has been carried out, and the results are given in Table 8.

The storage at the end of July is the maximum for which the reservoir need be designed. This volume, P_7, will be taken at 2 700 000 acre-ft.

The argument that the reservoir must be kept full at all times is manifestly illogical. The storage required at the end of March, for

Mr. Petterson.

TABLE 8.—PROBABLE MAXIMUM STORAGE* REQUIRED AT THE END OF EACH MONTH OF THE SEASON OF DEFICIENT FLOW, FOR THE 95% YEAR OF DRYNESS.

Draft = 0.5 Q'. Based on Curves of Fig. 9.

	ΣQ 95% year.	QUANTITIES, IN THOUSANDS OF ACRE-FEET.			
		ΣD	Storage, P, $= \Sigma D - \Sigma Q$	P	Required at end of:
(1)	(2)	(3)	(4)	(5)	(6)
ΣQ_9	3 550	6 221	2 700* 2 671	P_7	July.
ΣQ_8	2 900	5 515	2 615	P_8	August.
ΣQ_7	2 400	4 831	2 431	P_9	September.
ΣQ_6	2 000	4 125	2 125	P_{10}	October.
ΣQ_5	1 550	3 441	1 891	P_{11}	November.
ΣQ_4	1 250	2 735	1 485	P_{12}	December.
ΣQ_3	890	2 029	1 139	P_1	January.
ΣQ_2	670	1 390	720	P_2	February.
ΣQ_1	320	684	364	P_3	March.

* Figure adopted for maximum, 95% year.

instance, is only enough to tide over the low flow in April. The value, P_3, selected is such that it will be ample in 95 years of every century; and so on for other months, the maximum volume that need be in the reservoir is that tabulated in Column 4 of Table 8.

During what season of each year is this 2 700 000 acre-ft. of water to be impounded? In the 95% year, it is manifestly impossible to secure it during the interval from August 1st to April 30th, as the total run-off is less than the draft by just the amount of this storage. During the dryest years, then, the water must be impounded in May, June, and July, which are the months of replenishment. If the run-off of these 3 months is enough, in the 95% year, to maintain the rate of draft, D', and yet allow of replenishing the reservoir by 2 700 000 acre-ft., it will be more than enough to do this in wetter years.

It should be noted that the minimum flow for any month need not occur in the same year as the minimum flow for 2 or 3 consecutive months, which include the month in question. Also, the minimum run-off during the months of replenishment need not necessarily occur in the same water year as the one requiring maximum reservoir capacity.

The following procedure is then evidently on the safe side:

(1) From the records of run-off during July of each year, determine the probable minimum run-off of the 95% year.

(2) Do the same for the combined run-off of June and July.

(3) Do the same for the combined run-off of May, June and July.

The procedure is similar to that of Fig. 9.

Mr. Petterson.

Let the probable minimum values of the run-off, 95% year, be called:

$\Sigma\ Q_A$ for July.

$\Sigma\ Q_B$ " June and July.

$\Sigma\ Q_C$ " May, June, and July.

Also, let $\Sigma\ D_A$, $\Sigma\ D_B$, and $\Sigma\ D_C =$ the draft for the corresponding intervals.

MAGNITUDE OF SEASONAL RUN-OFF IN DIFFERENT WATER YEARS.

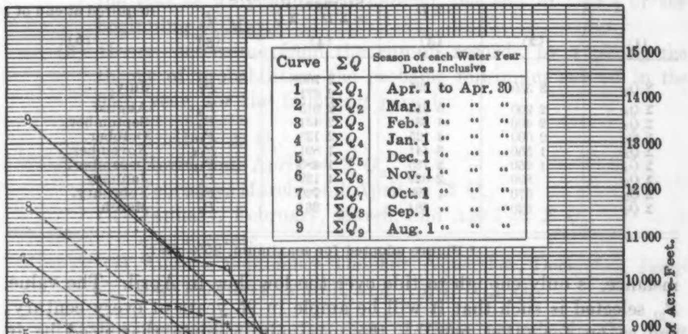


FIG. 9.

Then, $\Sigma\ Q_A - \Sigma\ D_A = P_A =$ storage available during July of the 95% year.

$\Sigma\ Q_B - \Sigma\ D_B = P_B =$ storage available during June and July of the 95% year.

$\Sigma\ Q_C - \Sigma\ D_C = P_C =$ storage available during May, June, and July of the 95% year.

On the assumption previously made, P_C must be equal to or greater than P_7 = the maximum reservoir capacity. Mr. Petterson.

Results were obtained as shown in Table 9.

TABLE 9.*

Months.	Run-off, 95% year, $= \Sigma Q$.	ΣD	Excess run-off = storage.	
May, June, and July	5 000† / 4 800	2 096	2 904 / 2 704	P_C
June and July	3 000	1 390	1 610	P_B
July	750	706	44	P_A

* Quantities in thousands of acre-feet.

† Probable minimum is in excess of quantity desired, so a smaller value is used.

Storage required in the reservoir by the end of July $= P_C = P_7$.

" " " " " " " " June $= P_C - P_A = P_6$.

" " " " " " " " May $= P_C - P_B = P_5$.

Case 1.—From the foregoing:

$$P_5 = P_C - P_B = 1\,094\,000 \text{ acre-ft.}$$
$$P_6 = P_C - P_A = 2\,660\,000 \text{ '' ''}$$
$$P_7 = P_C \quad = 2\,704\,000 \text{ '' ''}$$

P_C is actually 204 000 acre-ft. in excess of the required value of P_7, and a smaller value of ΣQ_C was used than its probable value for the 95% year.

The foregoing procedure, Case 1, determines the largest values of P_5 and P_6 necessary, and is based on a minimum flow for May, June, and July.

Case 2.—The run-off during May will not always permit of a storage of 1 090 000 acre-ft. during that month, as shown by figures in Table 10 based on the 95% year.

TABLE 10.—RUN-OFF AND STORAGE FOR 95% YEAR.
Quantities, in thousands of second-feet.

Months.	Run-off in 95% year, $= \Sigma Q$.	ΣD	Excess run-off = possible storage.	
May	1 600	706	894	P_5
May and June	3 850	1 390	2 460	P_6
May, June, and July	4 800	2 096	2 704	P_7

Mr. Petterson.

TABLE 11.—Comparison of Cases 1 and 2.

	Values of P, in Thousands of Acre-Feet.		Increment of Storage During Each Month.		
	Case 1.	Case 2.	Case 1.	Case 2.	Month.
P_5	1 094	894	1 094	894	May.
P_6	2 660	2 460	1 566	1 566	June.
P_7	2 704	2 704	44	244	July.

The apparent discrepancy is explained by the plausible supposition that the minimum run-off for May will not occur in the same year as the minimum run-off for July.

B.—Reservoir Capacity for High-Water Regulation Only.

As in the case of low-water regulation, the capacity of a reservoir for high-water regulation, or flood prevention, depends on the run-off and the rate of draft.

The draft for high-water regulation is the rate which can be discharged safely from the reservoir during the flood period. This rate will be called q. The maximum possible value of q is equal to the safe carrying capacity of the river channel below the reservoir. $\Sigma^T_0 (Q - q) = F$, the storage required for any flood.

Here T represents the duration of the flood, or the time in which Q is greater than q.

F obviously depends on the duration of high waters as well as on the intensity of flow. This is well illustrated in Figs. a and b of Plate XLVIII, where the assumed value of $q = 45\,000$ sec-ft. The volumes of F are proportional to the shaded areas in Fig. b. In Fig. a some of the floods of high intensity are seen to require a very small storage volume.

Seasonal Occurrence of Floods.—A great deal might be written on the seasonal occurrence of floods, but only one idea will be discussed.

Granting that floods may occur at any time on a particular river, it is practically universally true that there is a prevailing season when the maximum floods arise. This is particularly true if, by maximum floods, we mean those requiring the largest storage volume to regulate to safe flow, q.

In Table 12 is assembled the capacities, F, for all the floods during the 8 years of records. The storage is written under the month in which the flood began.

It will be seen that all the larger floods began some time in May. These larger floods are of long duration, and last well into July. Only the month in which the floods commence is of importance, however,

as will be seen later in the discussion of a reservoir for both low- and high-water regulation. Mr. Petterson.

The storage capacity required for the largest floods of each year is plotted on Fig. 10 for the purpose of determining the probable capacity, F_m, for the 95% year. From Fig. 10 it is taken as 7 150 000 acre-ft.

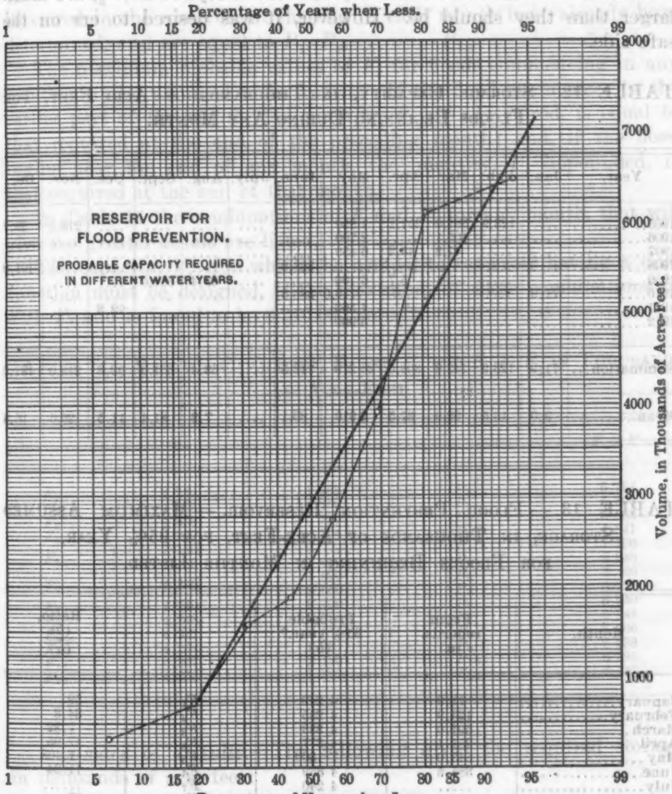


FIG. 10.

In the same way, the probable maximum capacities in the 95% year, for floods commencing in each of the months might be obtained. In the case of the data of Table 12, this procedure is not feasible because of the many blanks in the record, or years when no floods occur during such months.

Mr. Petterson.

As a rough approximation—which does not pretend to any reasonable degree of truth, but errs grossly on the side of safety—the curves for each month were assumed to be parallel to the regular line of Fig. 10, and to pass through the mean value in the 50% line. The quantities of Table 13 were thus obtained. There is no doubt that the values of F listed in Table 13, with the exception of F_5, are much larger than they should be. However, it was desired to err on the safe side.

TABLE 12.—STORAGE CAPACITY, IN THOUSANDS OF ACRE-FEET, FOR FLOODS BEGINNING DURING ANY MONTH.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1905	....	123.9	591.0	251.1	1 850	....	....	....	....	....	213.0	29.7
1906	....	....	105.1	....	2 700	....	....	....	....	....	....	.....
1907	....	....	47.2	43.2	6 450	....	....	....	....	....	....	.....
1908	....	....	....	....	......	328.0	....	....	....	....	....	101.9
1909	....	....	....	5.7	6 100	....	....	63.4	651.0	....	....	.....
1910	71.9	....	....	....	719.0	385.5	....	....	....	....	....	.....
1911	....	....	....	....	1 560	....	....	....	....	92.3	....	.....
1912	....	....	....	....	3 900	....	....	....	....	....	....	.....
Summation	71.9	123.9	743.3	300.0	23 279	713.5	....	63.4	651.0	92.3	213.0	131.6
Mean	9.0	15.5	92.9	37.5	2 910	89.1	....	7.9	81.4	11.5	26.6	16.4

TABLE 13.—FLOOD PREVENTION RESERVOIR.—MAXIMUM ASSUMED STORAGE, IN THOUSANDS OF ACRE-FEET, FOR 95% YEAR, FOR FLOODS BEGINNING IN MONTHS LISTED.

Month.	From records. (a)	Probable 95% year.* (b)		Ratio, $\frac{(b)}{(a)}$
January	71.9	4 249	F_1	59
February	123.9	4 255	F_2	34.3
March	591.0	4 333	F_3	7.32
April	251.1	4 277	F_4	17.05
May	645.0	7 150†	F_5	1.108
June	385.5	4 339	F_6	11.24
July	.....	4 240	F_7	......
August	63.4	4 248	F_8	67
September	651.0	4 321	F_9	6.64
October	92.3	4 252	F_{10}	46.1
November	213.0	4 267	F_{11}	20
December	101.9	4 256	F_{12}	41.8

* Figures for all months but May obtained by adding (7 150–2 910) = 4 240 to the mean quantities of Table 12, 4 240 = difference between maximum and mean of floods commencing in May.

† From Fig. 10.

Mr. Petterson.

C.—Reservoir Capacity for the Combined Function of Both High-Water and Low-Water Regulation.

If a reservoir had a quantity, P, impounded at the beginning of a flood flow, and a volume, F, were required to regulate the high water to safe flow, q, the combined capacity of the reservoir would evidently be $V = F + P$. Now, the required maximum capacities that need be stored in the reservoir at the end of each month have already been determined, and are equal to $P_1, P_2, \ldots P_{12}$.

The maximum probable values of F, for floods commencing in any month are also assumed to be known. If the flood commences in the earlier part of the month, the quantity, P, already stored, is equal to that required at the end of the preceding month; and, if the flood begins near the end of any month, the quantity, P, impounded, is that required at the end of that month.

In Table 14 the combinations of F and P, for any month, that will give the greater result are listed.

The capacity, V_c, for which the composite reservoir having a dual function must be designed, is the maximum of these combinations of $F + P$, and is found to be 8 240 000 acre-ft.

TABLE 14.—STORAGE CAPACITIES, F, AND P, AND $F + P$, IN THOUSANDS OF ACRE-FEET.

	F	P		$F + P$
F_1	4 249	1 485	P_{12}	5 734
F_2	4 255	1 159	P_1	5 394
F_3	4 333	720	P_2	5 053
F_4	4 277	364	P_3	4 641
F_5	7 150	....	P_4	7 150
	7 150	1 090	P_5	8 240
F_6	4 339	1 090	P_5	5 429
		2 660	P_6	6 999
F_7	4 240	2 700	P_7	6 940
F_8	4 248	2 700	P_7	6 948
F_9	4 321	2 615	P_8	6 936
F_{10}	4 252	2 431	P_9	6 683
F_{11}	4 267	2 125	P_{10}	6 392
F_{12}	4 256	1 891	P_{11}	6 147

Summary of Results.—The following gives the required storage, in thousands of acre-feet:

(A) Reservoir for low-water regulation only.... $= P_m = 2700$
(B) " " flood prevention only.......... $= F_m = 7150$
(C) " " combined function of A and B. $= V_c = 8240$
Sum of $P_m + F_m$.............................. $= 9850$
Saving in capacity by using composite reservoir.......... $= 1610$
or $V_c = F_m + 0.403 P_m$.

Mr. Petterson.

In general, $V_c = F_m + bP_m$, where b will always be less than 1 and frequently will be less than 0.5. Further generalizations cannot be made. The hydrologic data of each water-shed must be given individual study.

In case a larger rate of draft, D, were assumed, so that a yearly storage, P_y, is required, in addition to that for seasonal or monthly regulation, it would be added to the values of P_1, P_2, etc., so that P_m would be increased by the amount, P_y, as would also V_c.

If the draft, D', is increased greatly, it will be found that as D' approaches its limit, Q', the reservoir must of necessity act as a regulator of both high and low flows. It must store all excess run-off, in order to impound an adequate supply for the seasons of low water.

The volume, designated as U on Fig. *a* of Plate XLVIII, is that required to regulate to uniform flow, or where $D' = Q'$. This storage is nearly 19 000 000 acre-ft., or about seven times the storage necessary to regulate to a draft, $D' = 0.5\ Q'$, and 2.3 times the capacity determined for V_c, for combined flood prevention and low-water regulation to a draft of one-half of the mean flow.

With the lower rates of draft, the storage, P_m, is small, and F_m is relatively large, but, with the higher rates of draft, P_m becomes larger than F_m. Finally, F_m = zero and $P_m = V_c$, because all flood flows must be conserved in order to provide the water which is to sustain the draft.

D.—Operation of a Reservoir Having the Dual Function of Preventing Floods and Storing Water for Use.

The storage quantities computed in "A" are those required for the worst conditions assumed. In years of greater run-off a less quantity could be safely impounded, but wet and dry years cannot be safely predicted. No weather prediction whatever is needed, however. The reservoir operator is given instructions somewhat as follows:

Conserve all run-off during each month* of replenishment until the required volume of water for that month is impounded.

As soon as the required volume has been impounded, all excess inflow may be wasted.

The volume of water which must be in the reservoir by the end of each month has been determined before the reservoir was built. A table can be given to the operator, with these volumes expressed in height of water surface. He has only to observe water elevations on a graduated rod or gauge.

Thus, for the theoretical reservoir discussed herein: On May 1st commence conserving all excess run-off until the storage, P_5, has been

* These are May, June, and July for the stream studied. The particular months will always be known for any stream.

accumulated. If the volume, P_5, has been accumulated before the end of May, the remaining inflow may be wasted. There is no need of a greater volume before June 1st. Mr. Petterson.

During June conserve all excess run-off until a water level is reached corresponding to the storage, P_6. Maintain this level until July 1st.

Of course, if the level for storage, P_5, say, has been reached early in May and shortly afterward a heavy and prolonged run-off ensues, far in excess of maximum allowable outflow, the reservoir level will rise. If the flood flow is exceptional, the reservoir (of volume, V_c) may be completely filled; but such contingencies have been provided for in determining the volume, V_c.

The reservoir being assumed to be completely filled, or with water level higher than necessary for any month, it would be depleted at the rate,* $q + D'$, until the required level was reached.

The reservoir capacity, V_c, was designed for the 95% year. Floods may occur—at rare intervals—which would overtax this capacity. The writer believes that the spillway should be designed for such contingencies that the volume above the spillway overflow may exert its reductive effect after the reservoir has been filled; in other words, that flood prevention reservoirs, of the type discussed by the author, should be considered as a safety factor, and not as the main defense. The writer believes that "Impeding Reservoirs" would be a better name for basins of this class, with spillway outlets, to distinguish them from "Detention Basins" with orifice outlets. The ideal reservoir for flood prevention and storage for use would thus be a composite reservoir, consisting of:

1.—A lower volume, of capacity, P_m, which is a storage reservoir, but differing from previous conceptions, in that the water level does not need to be kept at a fixed elevation, but varies from month to month. Also, large controlled outlets should be provided to allow of wasting this water when desired.

2.—An upper volume, of capacity, $V_c - P_m$, with orifice outlets. This portion is a detention basin.

3.—The temporary storage above the overflow, provided as a factor of safety.

To repeat, in closing: The lower storage basin, of volume, P_m, allotted for low-water regulation, does not have to be kept full at all times, but can be operated on a definite schedule of water levels and time.

* Discharge, q, through wasteways or gates.
" D, through controlled outlets to conduit, or pipe lines.

Mr. Petterson. If the prevailing season for worst floods does not synchronize with a full storage basin, a considerable part of the volume, P_m, is available for storing flood waters, and the volume of the detention basin may be reduced accordingly. The capacity of the composite reservoir, therefore, may be considerably less than the combined volumes of two separate reservoirs, one of which is used for low-water regulation only, and the other is utilized for flood prevention only.

Mr. Houk. IVAN E. HOUK,* ASSOC. M. AM. SOC. C. E. (by letter).—The Engineering Profession is certainly much indebted to the late Gen. Chittenden for this, his last, contribution, as well as for his earlier writings.

As a general rule, the only feasible method of utilizing a detention basin for both water power and flood prevention seems to be that suggested by the author, namely, to use the capacity up to a certain elevation for storage purposes and above that elevation for flood prevention. Although cases may occur where automatic control of the part used for flood prevention will not be practicable, such as the one described by Mr. Meyer, the writer believes that no other method should be adopted except when absolutely necessary.

The writer does not believe that the design and operation of a detention basin such as proposed by Mr. Petterson, would be advisable. Although the paper by Allen Hazen, M. Am. Soc. C. E., on "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply"† is to be highly commended, the application of the theory contained therein to the design and operation of a detention reservoir for the purpose of flood prevention, as proposed by Mr. Petterson, does not seem to be justified. Such a study might be made as a matter of interest in investigating a project where stream-flow records are available for a considerable period, but the conclusion drawn by Mr. Petterson at the end of his discussion, namely, that "the capacity of the composite reservoir, therefore, may be considerably less than the combined volumes of two separate reservoirs, one of which is used for low-water regulation only, and the other is utilized for flood prevention only", as a rule, will not stand the test of application to particular cases.

In the example chosen by Mr. Petterson—the Colorado River at Yuma—most of the floods are caused by melting snows. Consequently, they occur at about the same season. Even in this case, however, it would not seem advisable to assume that the maximum storage will always be required in a certain month. If it should be required a month later than Mr. Petterson has assumed, the total required capacity would be 7 150 000 acre-ft. + 2 660 000 acre-ft., or 99.6% of

* Dayton, Ohio.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

RUN-OFF RECORDS, MIAMI RIVER, DAYTON, OHIO, 1893 TO 1917.

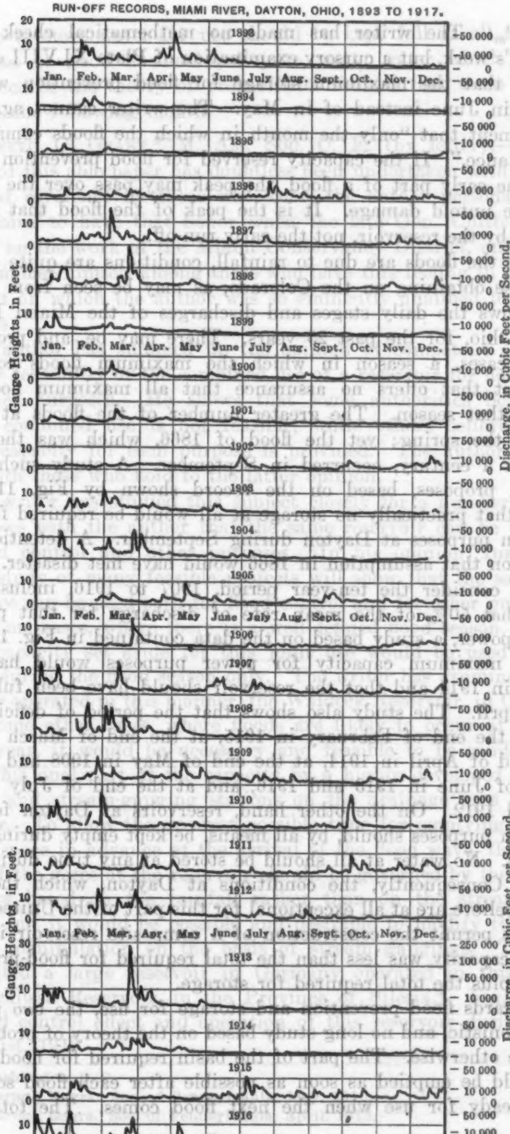


FIG. 11.

Mr. Houk.

$F_m + P_m$. The writer has made no mathematical check of Mr. Petterson's work, but a cursory examination of Plate XLVIII certainly indicates that the maximum storage for flood prevention would be required in June instead of in May. The writer cannot agree with the statement that "only the month in which the floods commence is of importance." If the capacity reserved for flood prevention is filled during the early part of a flood, the peak may pass over the spillway and cause untold damage. It is the peak of the flood that must be cared for by the reservoir, not the early run-off.

Where the floods are due to rainfall, conditions are quite different from those obtaining on the Colorado, as may be seen from Fig. 11, which shows the daily stages and discharges of the Miami River at Dayton, Ohio, for the past 25 years. There may be, and probably is in most cases, a season in which the maximum floods commonly occur; but that offers no assurance that all maximum floods will occur in that season. The greater number of the floods at Dayton occur in the spring; yet the flood of 1866, which was the second largest in a century, occurred in September. A study such as Mr. Petterson proposes, based on the record shown by Fig. 11, would indicate that practically no storage at all would be required for flood-prevention purposes at Dayton during September. A detention basin operated on that assumption in 1866 would have met disaster.

If we consider the ten-year period, 1907 to 1916, inclusive, and assume that 50% of the mean rate of discharge for that period is used for power, a study based on the data contained in Fig. 11, shows that the maximum capacity for power purposes would have been required in 1914 and that the reservoir should have been full at the end of April. The study also shows that the period of deficient flow began at the end of February in 1915, at the end of March in 1910, at the end of April in 1914, at the end of May in 1908 and 1911, at the end of June in 1913 and 1916, and at the end of July in 1907, 1909, and 1912. On the other hand, reservoirs at Dayton for flood-prevention purposes should, by all means, be kept empty during March and April. No water at all should be stored at any time during these months. Consequently, the conditions at Dayton, which the writer does not believe are at all exceptional for this part of the United States, would not permit the construction of a composite reservoir in which the total capacity was less than the total required for flood-prevention purposes plus the total required for storage.

As regards flood prevention and storage for use, the two purposes are antagonistic, and no long study based on the theory of probabilities can prove otherwise. The part of the basin required for flood prevention should be emptied as soon as possible after each flood so that it will be ready for use when the next flood comes. The total space

available to store water for use should be kept full at all times. It would not be good business policy to waste water which could be stored because the theory of probabilities showed that more would be available the following month. Mr. Houk.

Mr. Grant. KENNETH C. GRANT,* M. AM. SOC. C. E. (by letter).—The discussion of this able paper has doubtless been undertaken with a certain degree of reverence by those who knew the author personally. The writer came to have that honor when the late Gen. Chittenden was engaged on the work of the Miami Conservancy District, and can certainly number himself among those who have this feeling in discussing a subject on which the author was so eminently qualified to set forth his views.

The author has summed up the opinion of competent authorities as to the feasibility of the combined use of a reservoir for flood control and other purposes. He shows that some authorities believe that such a combined use is not feasible, while others believe it is, if storage capacity for each purpose is provided. The author takes his stand with those who hold to the latter opinion.

The writer has studied this subject closely for a number of years, and agrees with the author that the same reservoir can be used both for flood control and other purposes. In his opinion, furthermore, a close study of many feasible projects will show that the same reservoir space can be used both for flood control and for other purposes.

In some cases, where there is a well-defined flood season, that part of the capacity set aside for this joint use would be used for flood control during the flood season only. The writer has examined a large reservoir operated in this manner, the Waldeck Reservoir, in Germany.†

In some other cases, where floods are likely to occur at any time of the year, it would be necessary and feasible to be guided in the use of this dual part of the capacity by a highly developed system of rainfall and stream-gauging stations on the drainage area above the reservoir. To allow the greatest possible time for emptying this flood storage space in advance of the arrival of the flood wave, the manipulation of the gates at the reservoir should be governed by the estimates of inflow based on the rainfall rather than on the stream-gauging reports; but it would be possible and desirable to modify the original plan of manipulation on the basis of stream gaugings. The writer has visited a large reservoir in Germany operated in this manner, the Marklissa Reservoir, in the Province of Silesia.‡ Fig. 12 shows the flood control effect and operation of this reservoir during a flood in August, 1913.

* Washington, D. C.

† *Journal*, Engrs.' Soc. of Pennsylvania, May, 1916.

‡ *Journal*, Engrs.' Soc. of Pennsylvania, April, 1913.

Mr. Grant.

Such a combined use of the same storage capacity is especially practicable where two or more reservoirs are located one above the other on the same stream. Errors in prediction, which have led to manipulation of the outlets in the reservoir farthest up stream in such a way that the best results have not been obtained, can be corrected at the dams below.

There are many reservoirs in Europe which are used for both flood control and other purposes. Most of them are operated by keeping a certain fixed part of their capacity empty at all times and ready for the storage of damaging flood-water. In practically every case, this flood storage capacity is controlled by gates.

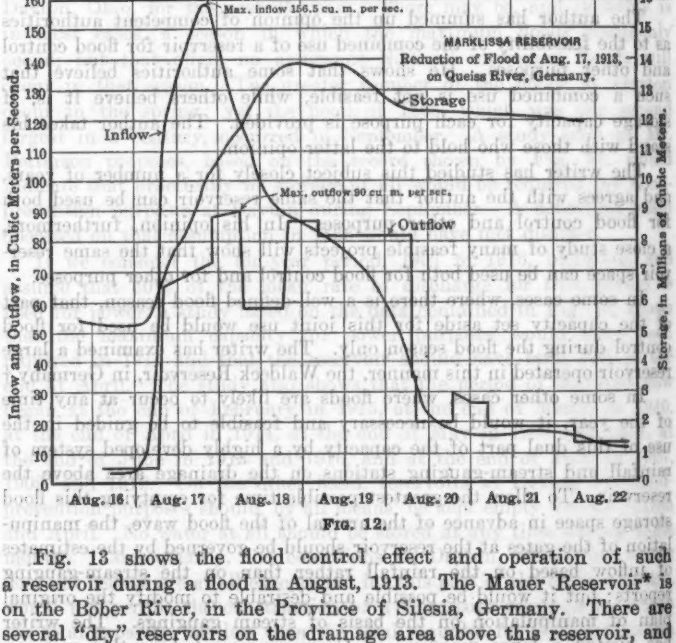


FIG. 12.

Fig. 13 shows the flood control effect and operation of such a reservoir during a flood in August, 1913. The Mauer Reservoir* is on the Bober River, in the Province of Silesia, Germany. There are several "dry" reservoirs on the drainage area above this reservoir, and their combined flood control effect is indicated by the difference between the estimated maximum inflow, assuming them not to be in operation, 475 cu. m. (16 770 cu. ft.) per sec., and the actual maximum inflow into the Mauer Reservoir, 287 cu. m. (10 130 cu. ft.) per sec., a reduction of 40 per cent.

* *Engineering News*, Vol. 69, p. 672 (April 3d, 1913).

In Europe, automatic control, as a rule, is used only in reservoirs built for flood prevention alone, the so-called "dry" reservoirs; though even this type is provided in some cases with mechanical control of the outlets. In most cases, the discharge through the dams of such reservoirs takes place through conduits or tunnels, rather than over spillways, the outlets often being placed at different elevations. Mr. Grant.

The author, in defining a detention reservoir, states that "the function is purely automatic, human control being wholly eliminated." The writer believes that the type of reservoir described by the author is just as truly a detention reservoir whether the outlets operate automatically or are provided with mechanical means of control.

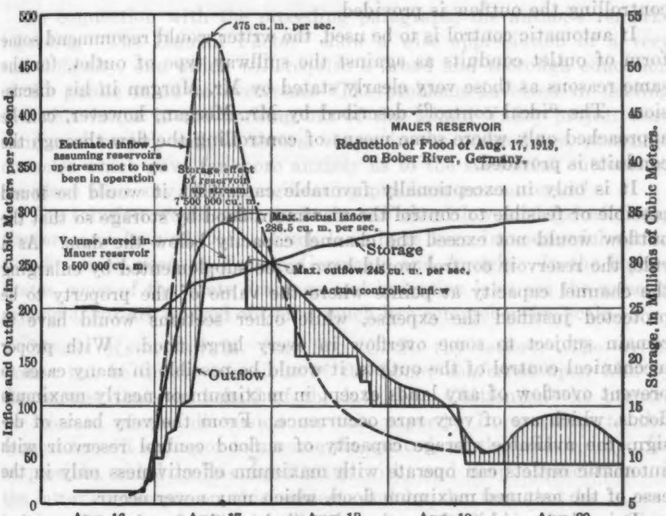


FIG. 13.

The writer fully appreciates the arguments in favor of automatic control of storage capacity set aside for flood control, whether in a reservoir used for both flood control and other purposes, or in a "dry" reservoir, particularly the former, where the temptation to encroach on the flood storage capacity may be very strong, and a long period of immunity from great floods may lull those in charge of operations into a false sense of security. He is coming more and more to believe, however, that it is possible to construct and maintain discharging apparatus which can be counted on to operate successfully at all times, and to train and keep on duty reliable operators who will obey orders. It

Mr. Grant. is also believed that the action of those higher up, who give the orders, can be controlled by suitable legislation and competent supervision.

With proper mechanical control of the outlets from a flood control reservoir, a given flood capacity can be operated with considerably greater effectiveness than with automatic control. With mechanical control, at the very start of a flood, water can be released from the reservoir at a rate equal to the carrying capacity of the channel below the dam, and thus less of the storage will be used in storing harmless flood water. Moreover, since any type of flood control reservoir must be designed for maximum flood conditions, it can operate at only part of its maximum efficiency in smaller floods, unless some mechanical means of controlling the outflow is provided.

If automatic control is to be used, the writer would recommend some form of outlet conduits as against the spillway type of outlet, for the same reasons as those very clearly stated by Mr. Morgan in his discussion. The "ideal control" described by Mr. Morgan, however, can be approached only where some means of controlling the flow through the conduits is provided.

It is only in exceptionally favorable cases that it would be found possible or feasible to control the maximum flood by storage so that the outflow would not exceed the channel capacity below the dam. As a rule, the reservoir control would have to be supplemented by enlarging the channel capacity at points where the value of the property to be protected justified the expense, while other sections would have to remain subject to some overflow in every large flood. With proper mechanical control of the outlets, it would be possible in many cases to prevent overflow of any lands except in maximum or nearly maximum floods, which are of very rare occurrence. From the very basis of design, the available storage capacity of a flood control reservoir with automatic outlets can operate with maximum effectiveness only in the case of the assumed maximum flood, which may never occur.

It is wisely said by the author that "it is of the first importance that a site once occupied be occupied to the full extent of its possibilities." As he says, "if occupied by an inadequate or inferior work, this may become a permanent bar to the highest development." If the time is not ripe for carrying out all parts of the project, the works should be designed so that additions for the other purposes can be made at a later date. Such a far-sighted utilization of available reservoir sites is possible only if two very essential steps are taken, namely:

(*a*).—The systematic collection and study on an intensive scale, by the proper agencies, of all data necessary for the solution of problems of water conservation and stream control, in order that all uses of the streams may be realized and co-ordinated. Without this informa-

tion, it will be impossible to determine what the full possibilities of a given reservoir site are. Mr. Grant.

(*b*).—The framing, passage, and active administration of suitable State and National legislation that will permit and foster co-operation between corporations, municipalities, States, and the Federal Government, in the construction and operation of storage reservoirs for all the various purposes for which they can be used. Without workable legislation of this kind, it will be impossible to bring together the various interests involved in the fullest utilization of our streams, or to compel the complete development of reservoir sites on which projects are proposed.

In connection with the preceding paragraph, the author's remarks regarding "The Human Factor" show a wise appreciation of a very real difficulty, one which will require a broad and thorough education of the public mind to overcome. The problems involved in dealing with this factor in the working out of the Miami Valley project were many and varied, and, at several stages in the proceedings, presented obstacles which gave far more anxiety as to the successful outcome of the plans than any of the engineering problems encountered.*

The solution would seem to be in a Federal law modeled after the Conservancy Law of Ohio. Such laws have been in successful operation for many years in France, Germany, and Austria. In the United States, some of the State drainage and irrigation laws are the same in principle, but are worked out on a much less comprehensive scale.

MORRIS KNOWLES,† M. AM. SOC. C. E. (by letter).—The writer cannot but be deeply conscious of the honor conferred upon him by the request that—in behalf of the late Gen. Chittenden—he prepare the closure of the discussion on this paper.‡ In acceding to this request he will endeavor to assume the attitude of the author, in so far as possible, aided by his familiarity with, and deep appreciation of, the latter's concepts on this subject, and by the fact that he is, and has long been, in accord with the aims expressed herein by Gen. Chittenden. The writer, however, is humbly impressed by his inability to interpret adequately the author's breadth of view and do justice to what has unexpectedly become a valedictory. The loss of Gen. Chittenden from the ranks of the Profession is indeed a serious one. It is to be regretted that so prolific a writer, with a mind so active and clear, should not have been permitted to be longer with us to take part in the development and witness the fruition of works that Mr. Knowles.

* *Journal*, Engrs.' Soc. of Pennsylvania, March, 1914, October, 1915, and November, 1916.

† Pittsburgh, Pa.

‡ This closing discussion on the paper by the late Gen. Chittenden has been prepared by Mr. Knowles in compliance with the request of the Publication Committee.—[SECRETARY].

Mr. Knowles. will undoubtedly be built in the near future in the attempt to solve this great problem of stream regulation.

Without doubt, Mr. Morgan has aptly caught and expressed Gen. Chittenden's purpose in presenting this paper, in that it is a plea to the Profession to "keep open a field of opportunity", in flood control, for the use of dual purpose reservoirs. The author's great openness of mind, as expressed by his willingness to change opinions when convinced after thorough study, is clearly stated by Mr. Morgan, in relating the circumstances surrounding the early investigations at Dayton, to which the writer can testify, for he was present as one of the group of "several other engineers"; and there is no better illustration of Gen. Chittenden's kindly, generous nature, which all who knew him learned to admire, than his statement on page 1485, when referring to the dual use of storage reservoirs:

"The writer would bespeak for the idea thus doubtfully referred to in the last two citations a more thorough and friendly consideration than has yet been given it. May not the superposition of one reservoir upon another contain important possibilities after all?"

To many, the consideration has become difficult and involved, because they have not kept clearly in mind the fact that dual use does not generally (and only in exceptional circumstances) contemplate control of the same volume of storage. In fact, as the author pointedly remarks, on page 1473:

"It is further generally recognized that flood control and storage for use are essentially antagonistic purposes which make impossible a reliable utilization of the same reservoir space for both."

It is well to remember that even the illustration of the Colorado River, so carefully and technically treated by Mr. Petterson, may be considered an exceptional instance, and not one of general application. Mr. Houk forcibly calls attention to this latter thought, in his analysis of the above mentioned discussion, with his illustration of the Miami River flood records. The author says, again, on page 1484:

"The question is not that of joint use of the same identical reservoir space, but that of having space for each purpose in addition to that required by the other. This, in fact, is the crux of the physical problem."

In the opinion of the writer, the "field of opportunity" seems so near to attainment, promises such obvious desiderata as to results, and such progress has been made toward its realization as a valuable method of flood control—both in theory and in the actual building of works—that to condemn the idea now, on the basis of our existing incomplete information and still more scant trial, would indeed be most unfortunate and ill considered. As the author has so well stated:

"Better by far to take some chances and register progress than to block progress because of contingencies which are so remote as to be practically negligible." Mr. Knowles.

The paper can in no sense of the word be regarded as a critique of "Detention Basins" or of "Reservoirs" as agencies for flood control. Indeed, in pointing out that the possibilities are not yet exhausted, the author has ingeniously proposed merely to superimpose a "detention basin" upon a reservoir—the former for purposes of flood control and the latter for such purposes as might otherwise be intended. Such a detention basin would then begin with the advantage of large area, as the greatest area of the reservoir is obviously its upper surface. Again, it would have all the advantages of the detention basin founded directly on the valley floor, as to conservation of land values within the flooded area, for it must be quite plain that the damage due to the land being flooded permanently by the reservoir should be charged against the purpose for which the reservoir is to be used—whether that purpose be the production of power, municipal water supply, whether for navigation or irrigation, or what not. The author expressly makes reservation, moreover, that he is not proposing a cure for all flood problems, but rather an application which may be useful in an "indeterminate zone of great extent" in harmonizing industrial and commercial uses, and flood control, in those cases where reservoirs can be made available for these purposes. Mr. Meyer's discussion of the very interesting case in point, of several uses at an unusually favorable site, seems quite apropos.

Nor can Gen. Chittenden's paper be considered a brief in favor of spillways *versus* conduits or orifices, in the automatic control of flood storage. The writer agrees with Mr. Morgan in that the author has purposely avoided the discussion of details—contenting himself simply with pointing out possibilities in substantiation of his original plea. Yet, are there not cases where spillways or spillway channels may automatically control the discharge of floods to the best advantage? Are not the relative merits of spillways, as contrasted with conduits for this purpose, affected to a major degree by the proportionate area of the reservoir in comparison with its water-shed? No one would think, for instance, of advocating conduits for flood storage in Lake Superior, to which Gen. Chittenden forcibly refers; and the writer can picture circumstances wherein artificial reservoirs of large area topped with a detention basin, as suggested, might be controlled to advantage simply by a spillway channel of proper proportions.

Mr. Petterson presents a most technical discussion of his thesis—that the same volume of storage may be used, in some instances, for different purposes, namely, to prevent floods and to increase stream flow at other times for valuable industrial purposes. It is sufficient to say that the detailed and rather abstruse mathematical work, as well as

Mr. Knowles. the basic data, in the paper* by Allen Hazen, M. Am. Soc. C. E., are of interest to the student of the subject, and are worthy of close study. Notwithstanding the reservation, which must be borne in mind, that there is danger in too general applicability, the calculations bear the evidence of correctness, and, though the premises are not broad enough to warrant so determinate a deduction, our judgment tells us that, with relatively uniform seasonal occurrence of floods, such use may be most advantageous.

Mr. Hall's interesting tabulation of the Ohio River floods is in line with Mr. Petterson's discussion of the Colorado project, namely, that dual use of some storage does not necessarily conflict in all cases. At least, according to the theory of probabilities, there is a large chance to use for many years much of the same storage space for both flood prevention and industrial purposes. The report of the Pittsburgh Flood Commission, published in April, 1912, shows, in a like manner, that 65.9% of the floods in the Ohio River, at the junction of the Allegheny and Monongahela, occur, during the years of record, in the months of January to March, inclusive, and that, during the remainder of any year, only one flood occurred exceeding 4 ft. above flood, or overflow, stage. No floods are recorded in September and October during the 105 years under observation.

Mr. Tibbetts calls attention to a somewhat different application of the detention basin principle due to the peculiar topographical conditions of the Sacramento Valley. His discussion of comparative costs and the use of appliances in different parts of the country, however, does not seem to be germane to the subject. The writer recalls that there were certain physical and hydraulic conditions which pointed toward the general inapplicability of the levee system in the Miami Valley, in addition to the problem of its excessive expense. The last sentence of Mr. Tibbett's discussion, however, expresses the truism: that each place and district has its own particular problem, governed by local conditions, demanding for each, therefore, a separate and distinct solution.

Capt. Grant refers to two important continental examples of the use of the "same reservoir space * * * for flood control and for other purposes." This is a valuable contribution to the discussion, presented in his usual clear and concise manner. The statement with regard to mechanical control *versus* automatic regulation is also pertinent, in showing that we must still keep our minds open, and study for the optimum means for each case or group of conditions. No greater truth has been presented than the following:

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

Mr. Knowles.

"If the time is not ripe for carrying out all parts of the project, the works should be designed so that additions for the other purposes can be made at a later date."

It is important that we should bear this in mind at the present time, when, under the stress of present conditions, we are inclined to do only that work which is essential and for which money can properly be utilized now. On the other hand, far-sighted vision demands that what is done now shall be of such basic design and strength that superimposed structures may be added later when the need of other uses makes it wise to build; and the essential steps of investigation, and study and collection of data, are important as a preliminary agency.

Gen. Chittenden's suggestion has served to prove his point, namely, that the field can by no means be considered closed to the successful use of dual purpose reservoirs—we must still keep an open mind. In this view, indeed, the writer concurs with the author, having expressed such a conviction in the Minority Report of the Special Committee on Floods and Flood Prevention.*

The subject of ways and means for the control of floods is by no means exhausted. In fact, there is still a paucity of information on fundamental phenomena. Floods and their control present a complexity of situations and conditions involving many different sciences, each a broad study in itself; and, as Gen. Chittenden has so often and so ably pointed out, not the least of these difficult conditions is the human factor. At best, it is not easy to impress the public mind with the advantages and importance of spending money on works for flood control and the necessary thorough, preliminary investigations; and any improvement or suggestion which will tend to harmonize the industrial use of works with flood prevention—to increase the dollar efficiency of the investment, even at the expense of a slight decrease in efficiency of flood control—will go far toward needed progress in this direction.

The discussion of the development of hydro-electric power at sites used for flood storage is particularly apropos at this time, when there is an unusual demand for electric energy, especially in industry where steam power is at present inadequate to furnish all the energy needed for war purposes, and at a time, also, when the difficulty of securing enough coal is a very pressing problem. National consideration of the development of water power becomes more prominent from day to day, and will continue to press for a broad and sane solution. Will not the possibilities of the dual use of reservoirs offer an inviting field, and make more promising the development of some sites formerly on the border line of financial desirability?

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1232.

Mr. Knowles.

Mr. Thomson strikingly refers to the enormous saving in coal that would accrue as a result of one proposed development. He also points to the fact that we do progress, and finally sift out the good in engineering practice, to which we may cling. The monetary discussion by Mr. Morgan, on page 1494, giving reasons for not considering it wise to add water power to the Miami Conservancy project, is interesting; but, with the changing values now and as time goes on, perhaps the balance will be sufficiently shifted to make such dual development worthy of consideration.

We are on the eve of a vast development and control of our natural resources, and, of these, water is one of the greatest. Unquestionably, much of the future greatness of America will be determined by the degree of wisdom with which we provide a comprehensive solution of this important problem of stream regulation so as to secure all the possible good and energy out of running water and control its devastating power. Frankly, the danger in the situation at present is the attempt to standardize on works or designs, in a field characterized by many remaining indefinite factors, after the remarkable controversies of years. Crystallization should not be attempted until opinions and data on the problem of reservoir control have become more concentrated and stable. As Gen. Chittenden wisely held, we would do well to postpone final conclusions until the evidence is complete, or, at least, has been more thoroughly collected and studied.

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Paper No. 1424

Discussion

FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By Messrs. L. J. Mensch, G. S. Bergendahl, E. S. Martin, W. K. Hatt, Charles F. Marsh, Clinton S. Bissell, J. R. Worcester, and F. E. Turneaure.

L. J. Mensch,† M. Am. Soc. C. E. (by letter).—The Final Report of the Special Committee on Concrete and Reinforced Concrete contains a great many excellent features, and, for its great devotion to the task allotted to it, the Committee deserves the thanks of the Society and of the general public, as all will benefit by its labors. It is to be regretted that sufficient funds were not at the command of the Committee so that it could have proceeded on the same scientific and practical lines as the French Committee in 1902 to 1905, or the German Committee from 1908 to 1915, or some other committees, the Special Committee of this Society on Steel Columns and Struts, for example. Mr. Mensch.

It seems that the Committee considered the tests made in various American colleges sufficient for the establishment of its rules. It nearly neglected the work of the French and German Committees, which solved many mooted questions. It was not unreasonable to expect that the Committee would have consulted these reports and would have made at least some new tests, in order to solve points left untouched by the others.

A great many of the Committee's rules do not contain any more information than may be found in the French report of 1905, and yet a great many more facts were known in 1916 than in 1905, and

* Previous discussions were published with the report as Paper No. 1398 in *Transactions*, Vol. LXXXI, p. 1101. The discussions which appear in this volume were received subsequently.

† Chicago, Ill.

Mr. Mensch.

would have permitted making more definite statements. Take, for example, the paragraph, "Freezing Weather." Tests could have been easily made, or reports from practice obtained, to give some more definite rules. It would have been very helpful to learn from the report how long to protect concrete in order to make it immune against freezing temperatures of various degrees.

In the chapter on "Forms" there is a very unfortunate paragraph, beginning, "Forms should be substantial and unyielding." All engineers know that lumber or steel deflects, and it would have been more proper to state the stresses and deflections for which forms are to be designed. The writer has never found that frozen concrete has a clear ring under the blow of the hammer, and the statement that this important test is unreliable ought to have been accompanied by the experimental facts.

The Committee recommends that, in columns, bars more than ¾ in. in diameter should be "properly squared and butted together in suitable sleeves." From the wording of this rule, one does not know whether to face the bars or merely hammer them square; neither would one know how long to make the sleeves. The fact is that it is practically impossible to obtain square ends or to place long bars on top of each other in order to bring their ends perfectly in contact; this, in practice, can only be obtained by a layer of cement which is poured into the sleeve before the upper bar is placed on the lower one, and tests show that a thin layer of cement grout is sufficient to produce a satisfactory transmission of stresses.

A great and important advance for American practice is the Committee's rule that the span length of continuous beams may be taken as the clear distance between faces of supports.

The assumptions recommended as a basis for calculation in Chapter VII are the same as those contained in the French report of 1905, and will hinder the clear understanding of the mechanics of reinforced concrete for another 10 years or more. The Committee states that it is well aware that these assumptions are not entirely borne out by experimental data, and that they are given in the interest of simplicity and uniformity. In Chapter X, working formulas based on these assumptions may be found, but engineers who are not very much in favor of concrete will not find these formulas sufficiently inviting for the adoption of reinforced concrete construction.

The writer is happy to state that the engineers of the building departments of our large cities have always been extremely courteous to him, and have never seen fit to compel him to undergo the ordeal of proving the safety of the structures he has submitted to them for approval by solving any formulas like those numbered (6), (11), (12), (16), (17), and (19). Formulas based on the conditions prevailing at the time of the ultimate load are considerably simpler and more

correct, because they agree with facts, while Formulas (6) to (19) agree with ancient theory. Mr. Mensch.

The writer would like to know the experimental facts on which the rules for the width of slabs in T-beams are based:

"(*a*) It shall not exceed one-fourth of the span length of the beam;

(*b*) Its overhanging width on either side of the web shall not exceed six times the thickness of the slab."

The tests of the French and German Committees show that one can count on a wider slab acting together with the stem.

Flat Slabs.—The Committee's ruling on bending moment coefficients is excellent, but the justification for it is rather lame. The Committee maintains with John R. Nichols, Assoc. M. Am. Soc. C. E., that the numerical sum of the positive and negative moments over the sections AF and GI of Fig. 2, disregarding the size of the column cap, is given by $\frac{W\,L}{8}$. The many tests which were made on buildings designed with bending moments with one-half this figure, or even less, compelled attention, and the Committee reduced this theoretical moment to an average of about $\frac{W\,L}{13}$ on account of the large size of column caps and because,

"Measurements of deformations in buildings under heavy load indicate the presence of considerable tensile resistance in the concrete, and the presence of this tensile resistance acts to decrease the intensity of the compressive stresses."

There is no reason whatever why, on these very same grounds, we should not allow a decrease of permissible moment coefficients for ordinary slabs, or slabs supported on four sides, or even T-beams with a small percentage of reinforcement. The statement that the presence of tensile resistance in the concrete acts to decrease the compressive stresses cannot be correct, because tensile stresses in the concrete mean less stress in the reinforcing; therefore, a reduction of the leverage of the centroid of tensile and compressive forces, and, for the same bending moment, will act to increase, and not to decrease, the compressive forces. If the Committee found by extensometer readings that the compressive strains are smaller than its theory warrants, then the proper deduction would have been that the theory was wrong and that the moments were actually smaller.

In his paper* Mr. John R. Nichols confessed that Grashof's formulas were a mystery to him, and maintained that from statics we are compelled to assume the numerical sum of positive and negative moments over one side and one center section as $\frac{W\,L}{8}$. The writer

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1670.

Mr. Mensch.

would like to know whether the Committee thinks that the great geniuses, Poisson, Saint-Venant, Grashof, Föpple, and Love, did not know statics? In the writings of any of these men will be found the following rules for finding stresses in an elastic body:

(1) Every particle of the inside of the body must be in equilibrium.
(2) Every particle of the bounding surface must be in equilibrium.
(3) There must be a proper relation between stresses and strains.
(4) There must be a proper relation between strain and displacement.

Grashof derived his formula, $\frac{W\,L}{16}$, by establishing the fact that every particle of his deformed plate was in equilibrium, and, therefore, of course, the whole plate; but we cannot invert such a proposition by saying that, because the plate as a whole is in equilibrium, every particle of the plate must be in equilibrium; because Mr. Nichols and the Committee overlooked this important proposition, their formula is wrong. Professor H. E. J. Love clearly states in one of his papers:

"If by any intuition we obtain the stresses in an elastic body, they must fulfill the above conditions, and great errors were made by neglecting to check the results with above conditions."

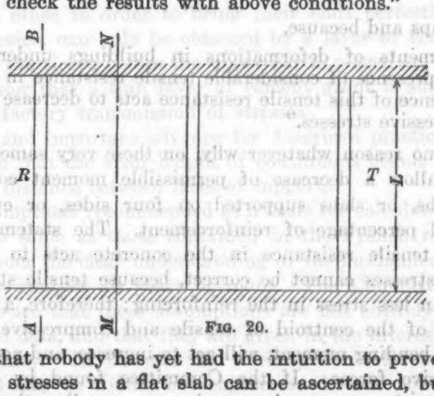


FIG. 20.

It seems that nobody has yet had the intuition to prove in a simple way how the stresses in a flat slab can be ascertained, but the writer will endeavor to show by a few propositions the absurdity of the indiscriminate application of statics (which apply only to an absolutely rigid body or to a body affected by such small forces that the strains and stresses are practically nil, and in which it does not make any difference whether we make an error of 2 or more) to an elastic body.

(*1*).—Fig. 20 is a plan of a number of freely supported girders connected by slabs and uniformly loaded by w per sq. ft. According to statics, the moment in Section RT is $\frac{wL^2}{8}$, and the numerical sum

of the positive and negative moments at AB and MN equals $\frac{w\,D^2}{8}$, Mr. Mensch.
where D is the distance of the girders. Assume now that the slab between the girders is very thick in comparison with the girder; according to statics, the value, $\frac{w\,D^2}{8}$, is still in force, but if the slab is strong enough to carry the load in the direction of L, we can, without fear, cut the slab at AB or MN, or in as many places in the direction of L as we please, and equilibrium will exist and the moments of $\frac{w\,D^2}{8}$ will vanish, even if D is very much longer or very much shorter than L. Hence we can make a static moment disappear without contravening any laws of Nature.

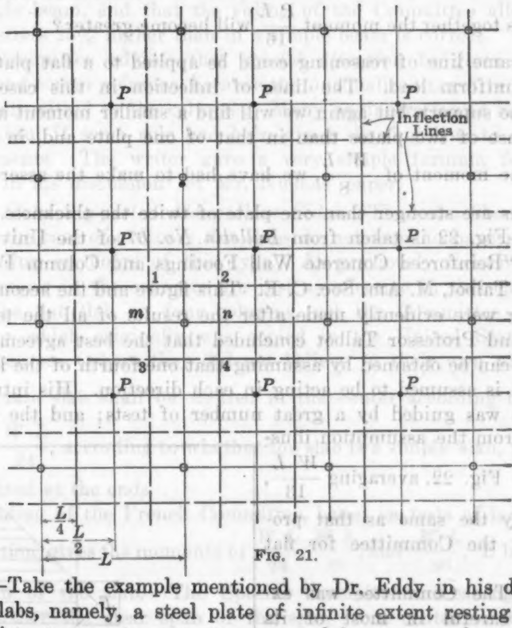


FIG. 21.

(2).—Take the example mentioned by Dr. Eddy in his discussion of flat slabs, namely, a steel plate of infinite extent resting on rows of equidistant separate supports, arranged in corners of squares, and loaded in the center of the squares by equal concentrated loads, P. (Fig. 21.)

On account of the perfectly symmetrical manner of loading, the inflection lines must be half way between the supports and the panel centers. For example, a square, 1, 2, 3, 4, if cut out of the plate, may be considered to be affected by shears on all four sides, each equal $\frac{P}{4}$,

Mr. Mensch. and by a concentrated load, P, acting upward at the center. The shears at each side are not distributed uniformly, but, assuming for a moment that they are equally distributed, then, according to statics, the moment about mn would be $\frac{P}{4} \times \frac{L}{4} + 2 \times \frac{P}{8} \times \frac{1}{2} \times \frac{L}{4} = \frac{3}{32} P\ L$. If now we make the further assumption that the plate, 1, 2, 3, 4, consists of two plates, each of half the thickness of the original plate, and that on one plate are acting the shears, 1, 2 and 3, 4, and on the other one the shears, 1, 3 and 2, 4, the moment about a line through 0 for each plate becomes $\frac{P\ L}{16}$. Will anybody dare to say that by welding the plates together the moment $\frac{P\ L}{16}$ will become greater?

The same line of reasoning could be applied to a flat plate loaded with a uniform load. The lines of inflection in this case will be nearer the support, but again we will find a smaller moment about mn in the case of two plates than in that of one plate and, in order to obtain the moment of $\frac{W\ L}{8}$, we have had to make the assertion that two plates are stronger than one plate of twice the thickness.

(3).—Fig. 22 is taken from *Bulletin No. 67* of the University of Illinois, "Reinforced Concrete Wall Footings and Column Footings", by A. N. Talbot, M. Am. Soc. C. E. This figure and the accompanying reasoning were evidently made after the results of all the tests were known, and Professor Talbot concluded that the best agreement with the tests can be obtained by assuming that one-fourth of the load (not one-half) is assumed to be acting in each direction. His intuition in this case was guided by a great number of tests; and the moment, derived from the assumption illustrated in Fig. 22, averaging $\frac{W\ L}{13}$, is exactly the same as that proposed by the Committee for flat slabs.

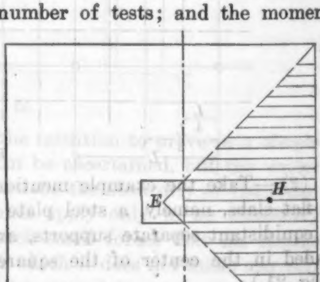


FIG. 22.

(4).—The Committee was extremely careful in most of its recommendations, and was always aware that any ruling it made must be made for average workmanship only; it was aware that only rarely is the designer the superintendent of construction (which ought to be the case), and

if this conservative Committee allowed the sum of the moments in flat slabs to be taken as $\frac{W\,L}{13}$ instead of $\frac{W\,L}{6}$ in beams or slabs, the numerical sum of the theoretical moments at one side and one center of one panel, for this reason alone, must be $\frac{W\,L}{16}$ rather than $\frac{W\,L}{8}$. Mr. Mensch.

(5).—It would have been more appropriate to point out the error in the books of Grashof, Föpple, or Lanza, than to lean on the tensile stresses of the concrete.

It is to be hoped that very soon proper tests will be made, proving that the so-called punching shear (which means shear near a support in a continuous structure) is not as dangerous as the ordinary shear in a simple beam, and that the ruling of the Committee allowing a shearing stress 25% higher than in a simple beam is correct.

The Committee advises that special attention be given to wall columns and corner columns, because the unequalized negative moment will be transmitted to the columns. Many engineers would have been very thankful if some rule had been given showing how to estimate such moments. The writer gave a very simple formula for these moments in his discussion* of Mr. Nichols' paper.

Floor Slabs Supported Along Four Sides.—The same liberal views shown in the case of ordinary flat slabs do not seem to have prevailed in the ruling on floors of this type. As previously mentioned, the tensile stresses in the concrete are just as effective in this case as in that of the flat slab on four columns, yet the Committee retained the ruling which originally was copied in the United States from German sources (where they did not believe in their own prophets), that a square slab shall be figured in the center according to $\frac{W\,L}{16}$, $\frac{W\,L}{20}$, or $\frac{W\,L}{24}$, according to whether the slab is a simple slab, partly or entirely fixed at the ends.

The ruling of the French Committee, based on tests of large slabs to destruction, gives the moments of $\frac{W\,L}{24}$, $\frac{W\,L}{30}$, and $\frac{W\,L}{36}$, L being the clear span of the slab. The Special Committee's meaning of the span—whether the clear span or the span from center to center of beams—is left indefinite in this case. The Committee evidently did not consult the tests to destruction on such slabs made by Professor Bach on behalf of the German Committee and others, or it would not have limited their use to cases where the length did not exceed one and one-half times the width. There are a number of tests on record where slabs of a length of two and one-half times the width

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1682.

Mr. Mensch.

showed an immense increase of strength over a slab supported only on the long sides.

The Committee's ruling is in full accordance with the building codes of nearly all large cities in the United States, and this unfavorable ruling tempts many designers to use flat slabs where they should not be used. The writer wishes to refer to concrete buildings from two to five stories in height, where, by the use of flat slabs, the stability of brick walls is endangered, or where, instead of the more economical brick walls, a skeleton construction is used in order to take care of the negative moments of the flat slabs in the outside panels. A more liberal ruling for slabs supported on four sides is urgently needed.

The chapters on diagonal tension, shear, and bond do not lay enough emphasis on the great importance of hooks at the ends of bars. Too much reliance is placed on the now nearly general use of deformed bars. The German Committee had tests made on about 100 beams, all of the same size, reinforced with the same percentage of steel, and all designed for the special purpose of finding the influence of hooks when only straight bars were used or when one-half or more of the bars were bent up, and in parallel series with and without stirrups. In this long series, not one beam can be cited where the omission of hooks did not show a marked reduction of the ultimate load over bars with large hooks. Where hooks were omitted in one-quarter of the bars, the drop was about 10%, where one-half of the bars were without hooks, the drop was about 20%, and where all the bars were without hooks, the drop was 50% and more. This importance of hooks in simple beams is greatly under-estimated in the United States, and wrong conclusions are often drawn from tests, because the great influence of hooks is overlooked.

Professor Moersch made tests on beams reinforced with deformed bars without hooks, and found the drop the same as with plain bars without hooks. The great importance of hooks is also shown in the tests made by Professor Talbot on wall and column footings. In the face of these facts, the paragraph: "As an additional safeguard, end anchorage may properly be used in special cases" and "anchorage of longitudinal bars at the ends of beams is advantageous," will not impress on the reader the importance of the subject.

It probably is a surprise to many engineers that the Committee advises the use of only two-thirds of the external shear in making calculations for the web reinforcing, yet this ruling is good engineering, and may be accounted for by the following consideration: A concrete beam has a considerable arch action, and may be considered to be a combination of truss systems like those shown in Figs. 23, 24, 25, and 26.

In a plain concrete beam, the truss system, *A*, will carry the same load as System *B*, for the reason that the width of the bands,

a and *b*, is probably very much greater than in the center of the beam, hence the stresses are very low and the strains in compression are the same as in tension. Mr. Mensch.

In a concrete beam with a low percentage of straight bars only, without stirrups (Fig. 23), the truss System *A* will carry a great deal more than System *B*, because the stresses in *a* and *b* are now very much higher, and *b* in tension is weaker than *a* in compression. In order that System *A* shall be effective, there must be a good connection between *a* and the horizontal reinforcement. This connection is generally furnished by the bond of the concrete to the steel rods, and when this bond is partly overcome, System *B* will take up a larger load until the stresses in *b* cause the concrete to crack, when System *A* will gradually take up the entire load.

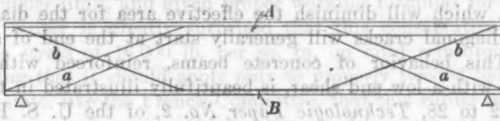


FIG. 23.

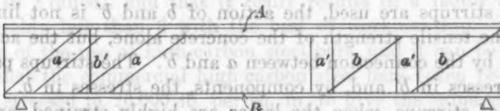


FIG. 24.

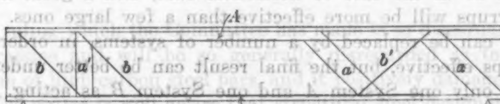


FIG. 25.

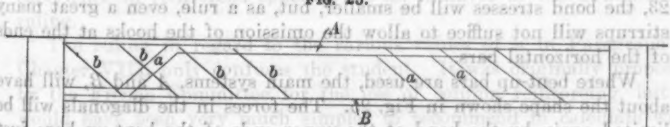


FIG. 26.

At this time, practically the entire horizontal component of the force in *a* must be taken up by the reinforcement near the support. The vertical component of *a* will have a tendency to bend the reinforcement, even to push it out of the concrete, and to strain the concrete in tension in a horizontal direction. In fact, many tests show horizontal cracks near the steel rods, which cracks will weaken the bond, and the connection between *a* and the steel rods will have to

Mr. Mensch. rely on the bond beyond the supports. Only rarely are freely supported girders extended out a sufficient length beyond the supports to form this connection, and the importance of hooks at the ends of the bars may now be seen clearly. Some of the tests of the German Committee seem to indicate that a large round hook in a 1-in. round bar is sufficient for an anchorage of 13 000 lb.

Where the shear in a concrete beam is low, the trusses will have a shape like that shown in Fig. 24. In this case, the diagonals, *a* and *b* are much steeper, and will require less bond than those in Fig. 23. The truss systems will be in action as long as the stresses, *b* and *b'*, are below the ultimate tensile strength of the concrete.

Beams are generally tested with concentrated loads, and vertical cracks in the concrete will appear near the points of application of the loads, which will diminish the effective area for the diagonals, *b*, and the diagonal cracks will generally start at the end of a vertical crack. This behavior of concrete beams, reinforced with straight rods, and with a low end shear, is beautifully illustrated in the photographs, 22 to 28, *Technologic Paper No. 2,* of the U. S. Bureau of Standards.

When stirrups are used, the action of *b* and *b'* is not limited any more to the tensile strength of the concrete alone, but the action of *b'* is limited by the connection between *a* and *b'*. The stirrups partly take up the stresses in *b'* and, by components, the stresses in *b*. Hooks at the ends of stirrups, when the latter are highly strained, are just as important as in the case of horizontal bars, and a great number of small stirrups will be more effective than a few large ones. Systems *A* and *B* can be replaced by a number of systems, in order to make all stirrups effective, but the final result can be better understood by assuming only one System *A* and one System *B* as acting. Because the horizontal component of *a* in Fig. 24 is smaller than that in Fig. 23, the bond stresses will be smaller, but, as a rule, even a great many stirrups will not suffice to allow the omission of the hooks at the ends of the horizontal bars.

Where bent-up bars are used, the main systems, *A* and *B,* will have about the shape shown in Fig. 25. The forces in the diagonals will be limited again by the bond at the upper ends of the bent-up bars just as *a* is limited by the bond at the ends of the horizontal bars. It may be noticed that the second bent-up bar from the support is in common for both systems *A* and *B,* which explains the fact that diagonal cracks generally appear first quite a distance from the support. Stirrups will be helpful, especially in the part where the bent-up bars are omitted, or near the supports, in order to prevent the weakening of the bond of the horizontal bars. In a well-designed beam, truss systems *A* and *B* will be equally effective, and each system will take up half the shear, but, evidently on account of the indeterminate action of the

trusses, the Committee decided on the rule to use two-thirds of the external shear for the calculation of shear members. That a graphical solution of stresses in shear members will be more correct than Formulas (22) to (27), is evident from the foregoing. Inasmuch as hooks play such a large rôle in the effectiveness of reinforcement, proper tests in this direction ought to be made at an early opportunity. Mr. Mensch.

In Chapter VIII, the writer finds a great advance in the ruling to allow 35% of the compressive strength in bearing, where the surface of the concrete is twice the loaded area.

The column ruling, although liberal and in full agreement with tests, is not broad enough for all practical applications. The Committee should not have placed the lower limit of spiral reinforcement at 1%; it should have been placed at ½%, and it should have allowed a higher upper limit, and given proper working formulas for these cases.

The writer is greatly surprised that this conservative Committee allows a stress of 16 000 lb. per sq. in. on mild reinforcing, without specifying that it should apply only to properly twisted bars, or otherwise to state, like the French Committee, that this ruling is on account of the demand of practice (high-carbon steel bars are not a commercial article in France), and that it really results in a factor of safety of considerably less than 3 with the best of workmanship. No mention is made of the use of high-carbon steel bars, and it can be stated positively that the commercial high-carbon steel bars (even re-rolled bars) can be stressed to 26 000 lb. per sq. in. to give the same strength in a reinforced concrete beam as mild steel bars stressed to 16 000 lb. per sq. in.

No ruling which the Committee has made is so discordant with general practice and will be so generally disregarded as the deliberate omission of high-carbon steel bars, and it positively damages the industry. The writer is well aware that structural steel designers with little experience in reinforced concrete are all in favor of the Committee's ruling.

The ruling in regard to the various values of n in Paragraph 8, Chapter VIII, only confuses the student. It was originally proposed by the French Committee for the use of column reinforcing, but it would have been very much simpler to recommend to calculate with an ultimate resistance of the steel rods of, say, 30 000 lb. per sq. in., which is, in fact, all that the various values of n result in.

G. S. Bergendahl,* M. Am. Soc. C. E. (by letter).—In the discussion of Professor Eddy's discussion by F. E. Turneaure, M. Am. Soc. S. E.,† it appears that Mr. Turneaure has not properly treated the shear in a flat plate floor. For example, in Fig. 15, he treats the shear on the column faces, C-B and D-E, as entering into the moment Mr. Bergendahl.

* Chicago, Ill.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI, p. 1199.

Mr. Bergendahl.

about *J-K*. Moment is a directed quantity, and shear enters into the production of moment only as it is normal thereto. When parallel thereto, it does not enter into the moment under consideration. Therefore, Mr. Turneaure has incorrectly taken twice the shear that actually operates in producing a moment normal to the line, *J-K*, and is in error in his criticism of Professor Eddy for that reason. This is elementary. Professor Talbot likewise appears to misunderstand this simple relation.

The Committee has correctly recognized this principle in its treatment of the square slab supported on beams on four sides in its division of the moment between the two directions, and it seems only thoughtlessness to have failed to follow the same correct principle in treating the shears on planes at right angles to each other, when the slab is supported on columns at the four corners instead of on beams at the four sides.

The report requires twice as much steel as that necessary to develop the strength of the concrete in flat slab construction. Therefore, 50% of the steel which the Committee's rule requires to be embedded, is wasted. It is strange that, with all the facilities this Committee has had at its disposal, it has not determined the relation of the quantity of steel that is required to balance and develop the full strength of the concrete in this type of construction. Any information which furnishes grounds for rational economy in engineering structures should, as Professor Talbot says, be welcome, even if it requires the frank admission of erroneous assumptions on the part of some of our members.

After building and testing flat slab floors for the past nine years, the writer has some very definite and exact information as to the deportment of structures of this kind in regard to which it would seem the Committee was entirely lacking, in view of the nature of its report. The advantages of this kind of construction can only be secured by the relatively close spacing of the slab rods. Those who have failed to carry out their construction with due consideration of this feature have secured results in keeping with beam action rather than the advantages of imitation of plate action.

The thing which is most striking about the Eddy theory is that by it one can, in a properly designed flat slab, actually predict and compute in advance the deflections that will be measured under load and the stresses that will be found in the steel. Substantial agreement of theory with ascertained fact may appear of no weight to Professor Talbot and certain members of the Joint Committee, but it is regarded by the practical engineer as the only criterion on which sound judgment can safely be founded.

E. S. Martin,* Assoc. M. Am. Soc. C. E. (by letter).—There are some recommendations in the report which merit critical consideration. Mr. Martin.

Under the head of "Floor Slabs Supported Along Four Sides", it provides that the reinforcement over the outer quarter slab widths may be one-half of that in the middle belt. This is a distribution which the writer has used many times, and believes to be conservative and well founded, but he fails to see any reason for using more reinforcement in the middle belt when the outer belts are reduced than when uniform reinforcement is provided. Certainly, the relief from flexure afforded to the slab along the supporting beams by these beams does not add to the bending moment sustained by the middle portion of the slab.

There is a requirement under Section 6 of Chapter VII, that reinforcement at the supports of continuous beams should extend beyond the inflection points a sufficient distance "to develop the requisite bond strength." This sentence, it is believed, will not generally be interpreted to mean exactly what it says, but, on the contrary, will be considered to require that laps of reinforced rods shall be sufficient to develop their full tensile strength at the point of inflection, notwithstanding that the tensile stress is zero at this point. Nearly all cases met in general building work are properly taken care of when the steel extends to the inflection points. For stubby, heavy beams, in which the tensile stress of the continuous steel piles up faster than the allowed bond provides for, the writer would prefer using a hooked end for anchorage rather than depend on tension being developed in the rod where it is surrounded by concrete in compression.

For "flat slabs", it is recommended that the capital slope be not more than 45° with the vertical; also, that the depression be four-tenths of the span in width and not more than one-half as thick as the slab. In a few cases the writer has designed floors with wide flat capitals, thinking that he had provided both capital and depression in one form. In this design he has assumed that the capital extended to the point where the total concrete thickness was twice the slab thickness. Although this design does not meet the requirements of the report, the writer is unconvinced that it is not good engineering practice—that it is not as good as that recommended by the Committee. The Committee's requirements are in danger of being followed blindly by those without independent judgment, and should not contain unnecessary provisions limiting freedom of design.

The method of slab design recommended—by inner and outer belts—is not the best that can be devised, although it has been used with different formulas in both the Philadelphia and Chicago rulings. The objection to this method is the fact that the critical moment and

* Philadelphia, Pa.

Mr. Martin.

TABLE 5.—(*Continued.*)

Building.	Panel and test.	Steel and slab.	Measured. Mid-span.	Measured. Capital.	American Society. Mid-span.	American Society. Capital.	Chicago. Mid-span.	Chicago. Capital.
Shredded Wheat Company, Niagara Falls.	20 ft. by 22 ft. 191 lb. 25-in. columns.	7 ½ in.; 8 ft. 6-in. capital. Drop, 8 ft. 6 in. by 2 in. Long, 10 ⅝-in. rods + 8 ½-in. rods. Long cant., 17 ⅝-in. rods + 8 ½-in. rods. Short, 10 ⅝-in. rods + 8 ½-in. rods. Short cant., 15 ⅝-in. rods + 8 ½ in. rods. Long mid., 12 ½-in. rods. Short mid., 10 ½-in. rods.	f_s 16 000 20 000 Wall panel 11 000 20 000 f_c 1 400 Int. 900 wall	f_s 16 000 .... f_c 1 400	15 900 25 300 540	18 100 690	....	....
Central Terminal Railway, Chicago.	24 ft. by 24 ft. 700 lb. 32-in. columns.	18 in. and 22 in.; 5 ft. 6-in. capital, Drop, 9 ft. 0 in. by 12 in. 20 ¾-in. rods, direct and diagonal, and over columns.	f_s 4 800 5 000 f_c 720 600	f_s 3 600 .... f_c 600	11 600 7 000 300	9 000 300	....	....
Schulze Bakery Company.	17 ft. 6 in. by 20 ft. 320 and 726 lb. 26- and 28-in. columns.	9 in. 4½-ft. capital. Drop, 7½ ft. by 5 in. 20 ⁷⁄₁₆-in. rods, diag. 23 " " " long. 17 " " " short. Cant. diag., doubled + ⅔ direct belts.	f_s 5 000 2 500 f_c 450 f_s 10 000 6 500 f_c 900	f_s 9 000 f_c 300 ...... f_s 10 000 ...... f_c 840	21 500 18 200 600 48 400 29 700 1 350	12 700 620 ...... 28 500 1 400	....	....
Curtis Leger Fixture Company.	17 ft. 10 in. by 19 ft. 200 and 400 lb.	8 in.; sl. cap. 4 ft. 6 in. 8 ½-in. sq. diag. 9 " " " short. 10 " " long. 10 " " square, all ways, wall.	f_s 3 500 7 000 f_c 18 000 7 000	f_s 4 000 f_c 400 f_s 12 000 f_c 900	15 400 11 200 30 800 22 400	10 200 500 20 400 1 000	....	....

Mr. Martin.

TABLE 5.

Building.	Panel and test.	Steel and slab.	Measured.		American Society.		Chicago.	
			Mid-span.	Capital.	Mid-span.	Capital.	Mid-span.	Capital.
Northwestern Glass Company, Minneapolis.	16 ft. by 17 ft. 400 lb.	8 in.; 4 ft. 6-in. capital. 15 ⅝-in. rods, 7 ft. wide. 8 1⅛-in. rods, radial.	f_s { 8 200 4 300	f_s { 22 000 20 000 17 500	25 800 20 900	21 500	25 800 14 400	28 800
			Wall panel { 12 000 15 000	f_c { 1 200 900	....	880	....	1 110
Deere and Webber, Minneapolis.	18 ft. 8 in. by 19 ft. 1 in. 340 lb.	9 9⁄16 in.; 4-ft. capital. 14 7⁄16-in. rods, diag. 12 7⁄16-in. rods, direct. 7 ft. 3 in., wide. 8 1¼-in. rods, radial.	f_s { 6 900 5 100	f_s { 20 700 18 300	26 600 19 500	19 500	26 400 19 000	27 700
			....	f_c { 795 750	....	750	....	990
Larkin Company, Chicago.	20 ft. by 24 ft. 2 in. 370 lb.	9 in.; 5-ft. capital. 8 ft., drop 6¾ in. 19 ½-in. rods, diag. 28 ½-in. rods, long. 18 ½-in. rods, short. 8 ⅞-in. rods, radial.	f_s { 3 300 3 000	f_s { 3 500 2 500	26 200 19 000	15 600	25 000 15 100	18 100
			f_c 550	f_c { 570	....	565	....	490
J Franks, Chicago.	19 ft. 4 in. by 20 ft. 3 in. 250 lb.	9 ¼ in.; 3 ft. 8-in. capital, drop 4 in. 18 ½-in. rods, diag., lapped. 16 ½-in. rods, direct. 9 ft. 6 in. wide.	f_s { 4 540 1 070	f_s { 4 575 3 440	18 800 8 000	7 300	12 700 7 500	8 050
			....	f_c { 677	....	310	....	335

Mr. Martin. stresses occur around the capital and decrease outward therefrom. To make the belt method at all workable, it is necessary to limit its application to designs having a fixed minimum ratio of capital diameter to span; otherwise one may find oneself relying on a belt of cantilever steel several times as wide as the capital, in which only the middle rods are really effective. A minimum limiting size of capital interferes with freedom of design, and unnecessarily so, because good designs, without over-stressing, may be made without capitals, if the slab is thick enough and contains sufficient steel.

The writer has made a comparison of the results found by applying the Committee's method to several buildings on which careful tests have been conducted. Detailed reports of these tests have been published in engineering journals.* For purposes of comparison, it is also of interest—and what is more, necessary—to examine the results of similar tests on beams, and beam and slab buildings. These figures are given in Tables 6 and 7. The flat slab figures are given in Table 5. Although the writer has not presented as many beam data as desirable, those given are sufficient for general conclusions.

In studying these figures, one should keep in mind the vital differences between the beam and slab floor and the flat slab floor. A glance at the computed and measured stresses for the increasing loads of Beam 72 (Table 6) shows that the tensile value of the concrete keeps the steel tension below the computed values for light loads, and that the concrete effect dies out with the heavy loads, loads causing measured stresses of from 16 000 to 20 000 lb. of unit tension in the steel. The percentage of tension steel (1.27) in this case is high. A lower percentage would have shown greater differences between computed and measured stresses. One can form no conclusions from the measured stress when this is low, but when it is high—from 16 000 to 20 000 lb.—the elongation is such that the concrete is broken up and its tensile effect destroyed. The percentage of reinforcement is not of much importance in such a case.

Now, in the beam and slab floors, it is evident that the entire slab is effective in tension at the supports of the beams and girders, and, consequently, the percentage of reinforcement (ratio of tensile steel to tensile concrete) is low, though the contrary is true at the mid-span of a beam. Such conditions do not exist in the flat slab, and allowances should be made. For instance, at the column capital of a flat slab floor, the maximum quantity of steel is used, and the only concrete in tension is that above the neutral axis of the slab for the circumference of the capital. The conditions as regards the tensile value of the concrete are similar to those at the mid-span of a beam.

Table 5 shows that the Committee method allows a liberal margin in all cases over the measured stresses in the reinforcement, but no

* Most of them are in the *Bulletins* of the University of Illinois.

Mr. Martin.

TABLE 6.

From *Bulletin 28*, University of Illinois, October 5th, 1908, Page 16. Beam 72. Percentage = 1.27

Measured.	Computed.	Measured.	Computed.	Measured.	Computed.
1 200	6 300	14 700	17 400	29 400	29 800
2 100	8 000	17 700	19 300	31 800	31 400
3 600	9 700	20 100	21 400	36 900	35 300
6 000	11 500	23 100	23 300	42 000	39 800
7 500	13 400	25 800	25 300	47 700	42 900
10 500	15 300	27 900	27 300	61 200	47 600
				108 000	49 600

TABLE 7.

			WENALDEN BUILDING. Stresses, 400-lb. test.		TURNER CARTER BUILDING. Stresses, 300-lb. test.	
			Measured.	Computed.	Measured.	Computed.
Girder.	End.	Steel...........	13 000	44 000	......	31 000
		Concrete........	2 200	1 700	900	1 200
	Mid-span.	Steel...........	17 000	19 000	9 300	12 500
		Concrete........	......	420	200	300
Intermediate beam.	End.	Steel...........	16 000	36 000	7 800	21 500
		Concrete........	2 000	1 900	1 200	1 300
	Mid-span.	Steel...........	16 000	22 000	9 000 (?)	18 500
		Concrete........	......	440	480 (?)	380
Column beam.	End.	Steel...........	11 000	69 000	......	19 600
		Concrete........	......	......	1 250	1 200
	Mid-span.	Steel...........	15 000	26 000	10 600	17 000
		Concrete........	......	......	380	330
			12 months old.		50 days old.	

Computed stresses: $\frac{W K}{12}$; $L =$ clear span + 3 in.

margin whatever for the concrete stress at the capital, although the latter is measured on an 8-in. gauge line, beginning at the capital, and is based on a nominal value for the modulus of elasticity. In the writer's opinion, the depression thickness, in general, should be not less than one-half the slab thickness, rather than not more than this, as a limit which the Committee states.

Mr. Martin.

Comparison of measured and computed stresses in the beam and slab floors shows evident arching, as the concrete stresses at the ends check roughly, and the steel stress at mid-span measures less than that computed, even at the higher values, and the concrete in tension is of too small a quantity to have much effect.

Several of the discussions on the report of the Committee have had reference to the theoretical phase. Although the writer has much respect for many of the theories put forth to explain the deportment of a flat slab, he does not consider that any of them so far advanced—not even the Committee's assumptions—have been established beyond question, so that they may be confidently followed in quantitative design, that is, to give definite values of stresses. He feels much safer when he knows that stresses are conservative, in the light of all the measured stresses of the numerous careful tests that have been conducted.

The writer questions the necessity of providing a wide margin of computed stress over the highest measured stress at the capital edge. The standard beam designs do not require a material margin at mid-span where corresponding conditions exist. He questions the necessity of providing a margin of from 300 to 500% at mid-span, except when the measured stresses are low, with low percentage. He questions the adequacy of the 20% additional for outside panels, if the interior panels are economically designed. Test measurements show that wall panels have stresses from 50 to 100% greater than interior panels, depending on the span length.

Mr. Hatt.

W. K. HATT,* M. AM. SOC. C. E. (by letter).—The Special Committee has adopted a simple and safe treatment of the mechanics of the reinforced concrete flat slab. The moment coefficients resulting from this treatment are thought to be too large by many constructors. Some attempt to reduce these coefficients by bringing into the analysis certain poorly defined phrases and novel elements that are without experimental basis. Others bring into evidence the large number of tests of flat slab floors in buildings, which uniformly show a resisting moment, as determined by steel stresses under design loads, which is much smaller than those specified by the Committee.

The writer wishes to record measured values of the internal moment, as measured by steel stresses for several tests, and to discuss the value of this evidence.

Such tests have usually been carried on by loading four panels of a floor to twice the designed live load plus the dead load, with measurements of the accompanying deformations in a portion of the steel bars. When the quantity and the depth of the steel are known, the internal resisting moment about the various sections may be computed.

The following records of tests are presented with a view to defining the extent of the discrepancy between the measured internal moment

* Lafayette, Ind.

and the external load moment. The discrepancy is not peculiar to flat slabs, but is found in reinforced concrete beams. Mr. Hatt.

A test of a single beam in Purdue Laboratory, with extensometer measurements of the steel, yielded the following:

Unit stress in steel.	Percentage of load moment in steel stresses.
2 600	16.5
16 300	74.4
21 800	81.5
27 200	86.5
39 300	95.5
44 700	98.1

The results of 333 tests of reinforced concrete beams with from 0.49 to 1.96% of reinforcement are given by Messrs. Humphrey and Losse.*

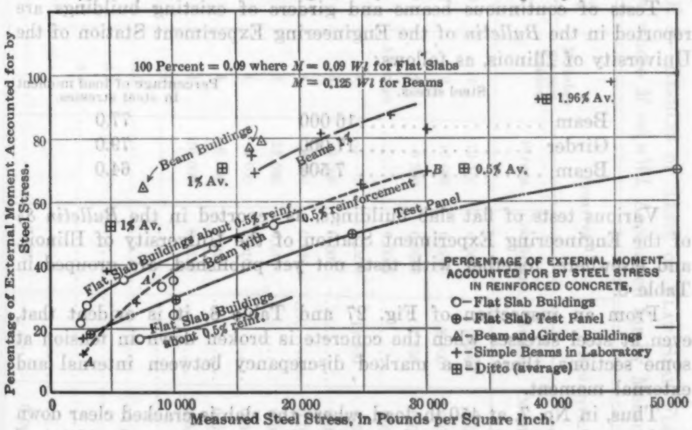


FIG. 27.

Those of the age of 4 weeks are platted on the diagram, Fig. 28, to show the amount of external moment accounted for by steel stresses. Evidently, as we might expect, the rôle played by the steel is less important for lower percentages. Thus, at stresses of from 30 000 to 44 000 lb. per sq. in.:

Percentage of reinforcement.	Percentage of external moment.
0.49	73
1.96	92

The line $AA'B$ in Fig. 27 shows the relation between stress and steel moment for two beams with a low percentage of steel (0.499).

* *Technologic Papers No. 2*, Bureau of Standards.

Mr. Hatt. A test of a sample beam in McGill Laboratory, with deformations of steel assumed to be the same as deformations of concrete measured on the surface of the beam, yielded the following*:

Unit stress in steel.	Percentage of load moment in steel stresses.
4 800	37.8
16 500	68.5
30 000	83.5
50 000	100.0

Eleven such tests at McGill showed an average percentage of load moment to be resident in steel stresses as follows:

5 000 lb. per sq. in. of steel stress, 52;
14 000 " " " " " " " 71.

Tests of continuous beams and girders of existing buildings are reported in the *Bulletin* of the Engineering Experiment Station of the University of Illinois, as follows:

	Steel stress.	Percentage of load moment in steel stresses.
Beam	16 000	77.0
Girder	17 000	79.0
Beam	7 500	64.0

Various tests of flat slab buildings, as reported in the *Bulletin 84* of the Engineering Experiment Station of the University of Illinois, and elsewhere, together with tests not yet published, are grouped in Table 8.

From an inspection of Fig. 27 and Table 8, it is evident that, even at steel stresses when the concrete is broken down in tension at some sections, there is a marked discrepancy between internal and external moment.

Thus, in No. 7, at 450 lb. load, where the slab is cracked clear down to the steel, which is stressed to 30 000 lb. per sq. in., only 46.4% of the external load moment, as fixed by Nichol's analysis, is accounted for. In this case there are no stress moments in corner columns to be added, and the load covered the structure completely.

Again, in Nos. 1-3, with a load of twice the live plus the dead load, with the concrete quite generally cracked, only 21.6% of the moment appears in the steel stresses. Here the columns at the corners of loaded panels contribute, and are not included in the steel moments, and the loaded areas are surrounded by unloaded panels.

As the steel stress increases, the percentage of moment resident in the steel increases also. As the percentage of reinforcement increases, the part played by the steel increases.

* *Transactions*, Can. Soc. C. E., Vol. XXII, p. 245.

Mr. Hatt.

TABLE 8.—Results of Tests of Flat Slab Floors.

Moment Coefficients, n, where $M \doteq nWl$, Accounted for by Measured Steel Stresses.

Building.	Load, in Pounds per Square Foot.		Number panels loaded.	Positive.		Negative.		Total Coefficient.	Percentage:	
	Design.	Test.		Stress.	Coefficient.	Stress.	Coefficient.		Theory.	Joint Committee.
(1) Franks	250	739	2	10 100	0.0068	9 200	0.0149	0.0217	24.0	28.0
(2) Larkin	250	738	2	16 000	0.0090	8 500	0.0135	0.0225	25.0	29.5
(3) Schultz	300	844	4	6 200	0.0038	7 100	0.0105	0.0143	15.8	19.0
			Average	....	0.0065	....	0.0130	0.0195	21.6	25.5
(4) Shredded Wheat	125	191	9	....	0.0210	18 000	0.026	0.0470	52.2	61.8
(5) *A*	150	150	4	2 500	0.007	....	0.013	0.0200	22.2	26.3
	....	300	4	9 000	0.010	....	0.020	0.0300	33.3	39.4
	....	400	4	13 500	0.013	....	0.028	0.0410	45.5	54.0
(6) *B*	150	150	4	3 000	0.0104	....	0.0133	0.0237	26.3	31.2
	....	300	4	6 000	0.0140	....	0.0180	0.0320	35.5	42.1
	....	400	4	10 000	0.0152	....	0.0176	0.0328	36.3	43.1
(7) Test slabs	150	150	All	....	....	....	....	0.0167	19.0	22.0
	....	300	All	....	....	....	....	0.0253	28.0	33.3
	....	450	All	....	....	....	....	0.0430	47.0	56.6
	....	600	All	....	....	....	....	0.0610	68.0	80.2
(8) Theory of Nichols ($c = 0.225\ l$)				....	....	....	....	0.090	....	....
(9) Joint Committee				....	0.029	....	0.047	0.076	....	....

Mr. Hatt. The evidence at hand points, probably, to a more persistent operation of these tensile stresses in the case of flat slabs, with about one-half of 1% reinforcement, than in beam structures, which are more heavily reinforced.

Although the curve for beams with a small percentage of reinforcing is not greatly different from that of flat slabs, there is this real difference of condition. Below A' the beam is not cracked in tension, but the flat slabs in this region of the diagram are cracked at

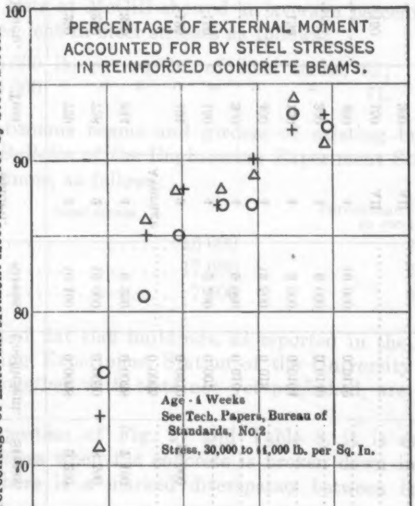


FIG. 28.

critical sections, at least down to the steel. There is also this difference, that, in the flat slabs, the deformations were measured on the steel, whereas, in the beams, they were measured on the surface of the concrete.

Individuals will no doubt attribute these remarkable discrepancies between steel internal moments, appearing in steel stresses, and external moments, to some other action than the tension in the concrete. The writer would rather look to the latter than to some poorly defined and unproved agency.

In either beam or flat slab structure, at ultimate loads producing a yield point in the steel and ultimate deformation in the concrete, the external moment and internal measured moment will agree. Mr. Hatt.

We turn to ask if these tensile stresses in the concrete, which thus appear to operate at high steel stresses under high loads, may not receive some credit by the designer—either in the form of reduced moment coefficients or increased steel stresses. The Committee has frankly stated that it is reasonable to allow for the large breadth of structure in a flat slab, and the fact that the tensile resistance of the concrete is less affected by cracks, and so the coefficient has been reduced from 0.09 to 0.076. Commercial practice has been using 0.066. It would appear that there is a comfortable factor of safety in the lower of these.

It must be remembered that, by reason of shrinkage, and by temperature changes, this tensile resistance of the concrete may largely disappear, especially in exposed structures. Likewise, the implications of the phenomena of plasticity are not well understood. Then, too, steel placed to take negative moment may be depressed by careless placing, or by the weight of workmen or wheel-barrows, and the leverage lost.

Of course, the concrete must react in compression to the steel stresses, and also to the tensile stresses in the concrete. It will not do to fix moment coefficients with respect to steel stresses only, for then the compressive stresses will be much larger than we compute them to be. However, it is probable that concrete in flexure, as ordinarily designed, has a larger factor of safety in the compression flange than in tension.

Considerations of mechanics, of workmanship, of experience in construction, and the use of flat slab buildings, should all enter into the regulations for design. Public safety should first of all be sought, but the conservation of material should also be kept in mind. A factory or warehouse is a tool of industry, and the designer will wish to produce a useful, economical, and satisfactory tool that is safe to use.

There are a number of unknown elements in flat slab design—questions relating to distribution, to lintel beams, and wall columns, to proportions of drop panels, to design of marginal steel. In view of the extent of the investment in flat slab floors, the necessity for economical construction in these critical times, and the probable competition in manufacturing after the war, it would seem necessary to spend the \$30 000 or \$40 000 required to build and test thirty flat slab panels. A saving of 10% in the cost of the flat slab floors constructed up to the present time, due to a more accurate knowledge of their action, would represent an amount of \$2 000 000.

The observation of deformations in the steel by skilled observers using the Berry extensometers is very accurate. In the earlier tests,

Mr. Hatt. the effect of changing temperatures on both the instruments and on the slab itself was not appreciated. The use of invar steel has done away with the changes of length of the instruments; but the changes in length of the reinforcing bars, due to changes of temperature of the surrounding concrete and the steel bars themselves, are important, especially at early loads. A change of 20° temperature has caused changes of length in the reinforcing bars equal to those occurring under the application of the load for which the floors were designed. Every precaution should be taken, therefore, to make the observations at such times as equal temperatures may be maintained, or to adjust properly the observations for temperature changes.

Mr. Marsh. CHARLES F. MARSH,* M. AM. SOC. C. E. (by letter).—In his suggested method of calculation for the design of reinforced concrete beams, Mr. Rhett† has fallen into the old error of equating the moments of the horizontal safe resistance.

It is hardly necessary to point out that such an equality seldom, if ever, exists in a reinforced concrete member.

The horizontal safe resistances in tension and compression must, of course, be equal to one another, and the couple formed by the action of these safe resistances must be equal to, or greater than, the imposed bending moment, but, unless the neutral axis is at the mid-point of the arm of the couple, it is obvious that the moments of the two resistances cannot equal one another.

Mr. Bissell. CLINTON S. BISSELL,‡ M. AM. SOC. C. E. (by letter).—In connection with the Final Report of the Special Committee on Concrete and Reinforced Concrete, the writer notes with interest the following statements in the discussion§ by Mr. A. H. Rhett:

"The Appendix to the report repeats the usual or standard textbook formulas for proportioning concrete beams. It has always seemed to the writer that though these formulas are normal for the analysis of a beam already built, they certainly operate in a reverse and roundabout manner for proportioning a new beam.

"One must first find p, a proportion of an unknown area, then K, a proportion of an unknown depth, then j, which is a fraction of the real depth, and then d; and, from d, one then works back to the absolute value of p.

"It is perfectly possible to solve a reinforced concrete beam as directly as a steel beam, and from the same basic elements, that is, the bending moment and the assumed fiber stresses and moduli of elasticity. * * *"

This criticism of the method of design is entirely justifiable, in the writer's mind. The method is one of approximation, and frequently

* London, E. C. 2, England.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI, pp. 1153–1159.

‡ Philadelphia, Pa.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1155.

leads to designs, the proportions of which are not in close conformity with the theory as presented. **Mr. Bissell.**

The standard notation and equations given in the Appendix to the report have been adhered to in what follows, and a few new symbols invented, such as are necessary to demonstrate a direct method for the design of reinforced beams and slabs. In the ideal beam, the resisting moment of compression, commonly called M_c, must equal the resisting moment of tension, commonly called M_s. This condition is realized by the use of the "steel ratio for balanced reinforcement" (Equation (5)), and by a fact which seems to have escaped notice heretofore, within the writer's observation, namely, when the two resisting moments are equal, the neutral axis of the beam is at a distance, k, from the compressive face of the beam such that:

$$k = \frac{n f_c}{f_s + n f_c} \quad \text{.......................(1)}$$

The report (Equation (5)) gives the ideal percentage as:

$$p = \frac{1}{2} \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{n f_c} + 1\right)}$$

Both equations apply in the case of beams reinforced for tension only, and also in the case of doubly reinforced beams.

For the moment, let the truth of these statements be assumed, and, referring again to the report (Equations (2) and (21)):

$$j = 1 - \frac{1}{3} k$$

$$f_s' = n f_c \frac{k - \frac{d'}{d}}{k}$$

It will be observed that, with one exception, these equations require for their solution only values for n, f_c, and f_s. The exception is the ratio, $\frac{d'}{d}$, in Equation (21). This is taken at $\frac{1}{10}$, provisionally, so that the practitioner may deduce k, p, j, and f_s', for any set of unit stresses and elastic ratio, n, which may be desired.

The compressive steel in a doubly reinforced beam is usually predetermined as a proportion, r, of the steel in tension, that is, $A' = rA$; for example, if A' is to be $\frac{1}{3}$ of A, then $r = \frac{1}{3}$. A new constant, G, will be used, involving r and the width, b, of the beam, in inches:

$$G = \frac{b\, p\, f_s\, f_s'}{\frac{f_s}{r} - f_s'} \quad \text{.......................(2)}$$

Mr. Bissell.

Two additional constants, K' and K, which probably are not new, will also be required:

$$K' = p\, b \qquad (3)$$

$$K = p\, b\, f_s\, j \qquad (4)$$

An elementary factor, m, of the bending moment, must also be determined. This is nothing more nor less than the bending moment of 1 in. of depth of beam; that is, if the concrete in the beam weighs 150 lb. per cu. ft., for example, and the beam is supported at each end, freely, over a span of, say, 20 ft., and is to be 10 in. wide, then the weight, W, of a 20-ft. element, 10 in. wide and 1 in. deep, will be:

$$\left(\frac{10}{12}\right) 150 \times 20 \left(\frac{1}{12}\right) = 208.33 \text{ lb.},$$

and, as the moment is $W\frac{l}{8}$, we have, for the elementary bending moment, $m = \dfrac{208.33 \times 20 \times 12}{8} = 6\,250$ in-lb. Had this beam been a cantilever, having a moment, $W\frac{l}{2}$, the element would have been $\dfrac{208.33 \times 20 \times 12}{2} = 25\,000$ in-lb. Hence, m = the bending moment, in inch-pounds, of an element of a beam 1 in. deep and b in. wide.

The total bending moment, M, which a beam is required to sustain, consists of the moments of all weights and allowances due to live load, impact, road paving, tracks, etc., including that due to the weight of the beam itself. This latter moment is evidently equal to $m\,(d + d'')$, in which d'' is the distance from the center of the tensile steel to the tension face of the beam, and the remaining part of the total moment, M, may be designated as the external moment, M_E, all parts of which are specified or otherwise known. Hence,

M_E = the external bending moment, in inch-pounds, which the beam must sustain in addition to the moment due to the weight of the beam itself.

The final equations for the design of beams and slabs may be readily derived; they are as follows:

For tensile reinforcement only:

$$d = \sqrt{\frac{M_E + m\,d''}{K} + \left(\frac{m}{2\,K}\right)^2} + \frac{m}{2\,K} \qquad (5)$$

$$\left.\begin{aligned} A &= K'\,d; \text{ and } M = M_E + m\,(d + d'') \\ \text{For check:}\quad d &= \sqrt{\frac{M}{K}} \end{aligned}\right\} \qquad (6)$$

Mr. Bissell.

For doubly reinforced beams:

Case I.—When the depth of the beam is unlimited:

$$d = \sqrt{\frac{M_E + m\,d''}{K + 0.9\,G} + \left(\frac{m}{2\,(K + 0.9\,G)}\right)^2} + \frac{m}{2\,(K + 0.9\,G)} \quad \ldots (7)$$

which gives d when the ratio, $\frac{d'}{d} = \frac{1}{10}$.

$$d = \sqrt{\frac{M_E + m\,d''}{K + G} + \left(\frac{m + G\,d'}{2\,(K + G)}\right)^2} + \frac{m + G\,d'}{2\,(K + G)} \quad \ldots (8)$$

$$A' = \frac{G\,d}{f_s'};\ A = \frac{A'}{R};\ \text{and}\ M = M_E + m\,(d + d'') \quad \ldots (9)$$

Case II.—When the depth of the beam is limited:

$$X = \frac{M - K\,d^2}{f_s\,(d - d')} \quad \ldots (10)$$

$$A = X + K'\,d;\ A' = \frac{X\,f_s}{f_s'\,(d - d')};\ \text{and}\ r = \frac{A'}{A} \quad \ldots (11)$$

All the constants in these equations may be deduced by Equations (1) to (4), inclusive.

The working equations resulting from the introduction of the known values used for any given case are simple, and easy of application. It is found convenient to assume a width, b, of 12 in. in forming the equations, as it is the usual width of an element for slabs. Furthermore, the external bending moment, M_E, can always be reduced to an equivalent for 12 in. of width, as it is directly proportional, that is, a moment of 800 000 in-lb., for a beam 8 in. wide, would be 100 000 in-lb. per inch of width, or, 1 200 000 in-lb., for a beam 12 in. wide. The areas of steel reinforcement determined for the 12-in. beam must be multiplied by $\frac{8}{12}$, in order to give the proper value for the 8-in. beam. The same reduction must be made for the total moment, M.

Assuming that the working values are to be $b = 12$, $n = 15$, $f_s = 16\,000$, and $f_c = 650$, we have from the foregoing equations:

For tensile reinforcement only (Equations (5) and (6)):

$$d = \sqrt{\frac{M_E + m\,d''}{1\,290} + \left(\frac{m}{2\,580}\right)^2} + \frac{m}{2\,580} \quad \ldots (a)$$

$$A = 0.0923 \times d \quad \ldots (b)$$

$$M = M_E + m\,(d + d'');\ \text{for check},\ d = \sqrt{\frac{M}{1\,290}} \quad \ldots (c)$$

Mr. Bissell.

For double reinforcement (Equations (2), (7), (8), and (9)):

Case I.—When d is unlimited:

$$G = \frac{10\,595\,400}{\frac{16\,000}{r} - 7\,175} \dots\dots (d)$$

$$d = \sqrt{\frac{M_E + m\,d''}{1\,290 + 0.9\,G} + \left(\frac{m}{2\,(1\,290 + 0.9\,G)}\right)^2} + \frac{m}{2\,(1\,290 + 0.9\,G)} \dots (e)$$

$$A' = \frac{d\,G}{7\,175};\ A = \frac{A'}{r};\ \text{and } M = M_E + m\,(d + d'') \dots\dots (f)$$

These five equations determine the beam when the ratio of $\frac{d'}{d} = \frac{1}{10}$; as d is now known, the ratio, $\frac{d'}{d}$, may be found for any desired value of d', and the following equations apply:

$$f_s' = 9\,750\,\frac{0.3786 - \frac{d'}{d}}{0.3786} \dots\dots (g)$$

$$G = \frac{1\,477 \times f_s'}{\frac{16\,000}{r} - f_s'} \dots\dots (h)$$

$$d = \sqrt{\frac{M_E + m\,d''}{1\,290 + G} + \left(\frac{m + G\,d'}{2\,(1\,290 + G)}\right)^2} + \frac{m + G\,d'}{2\,(1\,290 + G)} \dots (i)$$

$$A' = \frac{d\,G}{f_s'};\ A = \frac{A'}{r};\ \text{and } M = M_E + m\,(d + d'') \dots\dots (j)$$

Case II.—When d is limited (Equations (10) and (11)):

The total bending moment, M, in this case is known:

$$X = \frac{M - 1\,290\,d^2}{16\,000\,(d - d')} \dots\dots (k)$$

$$\left.\begin{aligned} &A = X + 0.0932 \times d;\ A' = \frac{X \times 16\,000}{f_s'};\\ \text{or,}\quad &A' = \frac{X}{1.61\left(0.3786 - \frac{d'}{d}\right)} \end{aligned}\right\} \dots\dots (l)$$

To illustrate the use of these equations by working the same example under all three cases, let it be assumed that a beam is to be designed, having a span of 24 ft.; that the beam is freely supported at each end $\left(\text{that is, the moment of its weight will be } W\,\frac{1}{8}\right)$; and

that the material in the beam weighs 150 lb. per cu. ft., the remaining data being as follows: $f_s = 16\,000$; $f_c = 650$; $n = 15$; $b = 12$ in., as above; $d'' = 3$ in.; and $M_E = 900\,000$ in-lb. For this beam, Mr. Bissell.

$$m = \frac{1 \times 24 \times 150 \times 24 \times 12}{12 \times 8} = 10\,800 \text{ in-lb.}$$

Example 1.—For tensile reinforcement only, applying Equations (*a*), (*b*), and (*c*):

$$d = \sqrt{\frac{900\,000 + (10\,800 \times 3)}{1\,290} + \left(\frac{10\,800}{2\,580}\right)^2} + \frac{10\,800}{2\,580} = 31.4 \text{ in.}$$

$$A = 0.0923 \times 31.4 = 2.90 \text{ sq. in.}$$

$$M = 900\,000 + 10\,800\,(31.4 + 3) = 1\,271\,520 \text{ in-lb.}$$

Check: $d = \sqrt{\dfrac{1\,271\,520}{1\,290}} = 31.4$ in.

Example 2.—For double reinforcement in which $d' = 2.5$ in., and $A' = \frac{1}{3}A$, or $r = \frac{1}{3}$, the other known values being the same as in Example 1.

Applying Equations (*d*) and (*e*):

$$G = \frac{10\,595\,400}{\dfrac{16\,000}{\frac{1}{3}} - 7\,175} = 259.53$$

$$d = \sqrt{\frac{932\,400}{1\,524} + \left(\frac{10\,800}{3\,047}\right)^2} + \frac{10\,800}{3\,047} = 28.42 \text{ in.}$$

This, however, is for a beam in which $\dfrac{d'}{d} = \dfrac{1}{10}$, or $d' = 2.84$ in. Ordinarily, this determination will be sufficient, and Equation (*f*) may be used:

$$A' = \frac{28.42 \times 259.53}{7\,175} = 1.028 \text{ sq. in.}; \quad A = \frac{A'}{\frac{1}{3}} = 3.084 \text{ sq. in.};$$

$$M = 900\,000 + 10\,800\,(28.42 + 3) = 1\,239\,340 \text{ in-lb.}$$

To be more exact, however: the ratio, $\dfrac{d'}{d} = \dfrac{2.5}{28.42} = 0.0880$, and, applying Equations (*g*), (*h*), (*i*), and (*j*):

Mr. Bissell. $f_s' = 9\,750 \dfrac{0.2906}{0.3786} = 7\,483.7$ lb. per sq. in.; $G = \dfrac{1\,477 \times 7\,483.7}{40\,516.3} = 272.82$

$$d = \sqrt{\frac{932\,400}{1\,563} + \left(\frac{11\,482}{3\,126}\right)^2} + \frac{11\,482}{3\,126} = 28.37 \text{ in.}$$

$$A' = \frac{28.37 \times 272.82}{7\,483.7} = 1.034 \text{ sq. in.}; \; A = \frac{A'}{\frac{1}{3}} = 3.103 \text{ sq. in.}$$

$$M = 900\,000 + 10\,800\,(28.37 + 3) = 1\,238\,800 \text{ in-lb.}$$

A second exact method is to use the value, $d = 28.42$, and the moment, 1 239 340, in Case II, where d is limited. In this case, the ratio, r, will be slightly affected:

Example 3.—Case II, in which d is to be 28.37 in., the other known values to be the same as in Example 2; here, the total moment, M, is 1 238 800 in-lb.; applying Equations (k) and (l):

$$X = \frac{1\,238\,800 - 1\,290\,(28.37)^2}{16\,000\,(28.37 - 2.5)} = 0.4845$$

$$A = 0.4845 + 0.0932 \times 28.37 = 3.103 \text{ sq. in.}$$

$$A' = \frac{0.4845}{1.61\,(0.3786 - 0.0880)} = 1.034 \text{ sq. in.}$$

which agree with the values in Example 2.

It will be noted that the proportions derived by these equations are adequate to sustain both the external moment and the moment due to the weight of the beam itself.

It will also be found on trial that the resulting proportions agree with the equations given in the report, which goes far to prove the truth of the statements made by the writer in introducing the subject.

The mathematical proof is to be found by equating the moments about the neutral axis, and combining these results with the linear proportions of the deformation diagram, which involve the moduli of elasticity.

It will also be observed that the writer's Equation (1):

$$k = \frac{n\,f_c}{f_s + n\,f_c},$$

which expresses this relation, is the same as Equation (20) in the report.

The use of these equations affords a direct solution in each of the cases mentioned, and the results obtained will be found to be in exact conformity with the equations cited in the report.

Mr. Worcester.

J. R. WORCESTER,* M. AM. SOC. C. E. (by letter).—Mr. Bissell has approved the point in the discussion† by A. H. Rhett, Assoc. M. Am. Soc. C. E., which is based on a fundamental error, namely, that in the "ideal beam, the resisting moment of compression * * * must equal the resisting moment in tension." He also states that "this condition is realized by the use of the 'steel ratio for balanced reinforcement.'"

The first of these statements can be easily disproved by the following considerations:

If the moment of compression is equal to the moment of tension, as the total compressive stress must equal the total tensile stress, the lever arms, from the neutral axis to the centroid of compression, must equal that from the neutral axis to the center of the reinforcement. This is the case only where $K = \frac{3}{5}$ and $\frac{F_s}{F_c} = \frac{2}{3}\,n$. Such a ratio is highly inconsistent with "balanced reinforcement" and with the stresses assumed by Mr. Bissell in his examples.

The second statement, as to "balanced reinforcement", is due clearly to a misunderstanding of the term. This might have been made clearer in the report of the Committee. It merely means a sufficient quantity of reinforcement, so that the stress in the steel shall bear the same ratio to the maximum stress in the concrete as the maximum stresses in the two materials allowed by the specifications. This condition, although it leads to using the two materials economically, is by no means generally desirable. Many conditions may make it desirable to use less steel than the maximum which might be allowed. Generally speaking, the stress in the steel is what governs the design, rather than the stress in the concrete; that is, the concrete will be stressed to much less in compression than it might safely carry; but, on the other hand, it not infrequently occurs that the stress in the concrete will be the controlling feature, and that an excess of steel will be desirable.

Mr. Turneaure.

F. E. TURNEAURE,‡ M. AM. SOC. C. E. (by letter).—The discussion of the report of the Joint Committee on Concrete and Reinforced Concrete has, perhaps, already been too lengthy, but, in view of the criticisms of the report relative to some very fundamental matters, it seems desirable to make a further reply, from the standpoint of the statical analysis used by the Committee.

In the discussion by Messrs. Mensch and Bergendahl, the Committee's analysis is questioned on the theory of "directed shear". In the discussion on page 1199,§ the writer has applied the simple laws

* Boston, Mass.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1155.

‡ Madison, Wis.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXI.

Mr. Turneaure.

of statics to the half slab, and has obtained values for bending moments in the slab by taking into account all the vertical reactions or shears at the corners of the slab, whether or not these shears act on lines parallel or at right angles to the direction of the bending moment. The critics of this method of analysis claim that reactions or shears acting on planes parallel to the direction in which the bending moment is calculated have no effect thereon. In other words, it is claimed that the principles of statics are not applicable to this case. It is plain that if the shears on *BC* and *ED*, Fig. 15, page 1200,* are omitted in determining bending moments on *JK* and *CD*, the equation of equilibrium between internal and external moments will result in no bending moment at all on these lines, because the moment of the shears on *AB* and *EF*, amounting to $2 \times \frac{1}{8} W \times \frac{L}{2} - \frac{1}{2} W \times \frac{L}{4}$, would equal zero. This sort of process is entirely inexplicable to the writer. If the principles of statics are to be applied to certain cases and rejected in others, it is difficult to say where to draw the line. Perhaps the remarks of Mr. Mensch will serve to determine this point. He states, on page 1544, that the laws of statics may be applied "only to an absolutely rigid body or to a body affected by such small forces that the strains and stresses are practically nil." This is an astonishing statement, and, if true, would render engineers practically helpless in all stress calculations. If it is true that reaction shears such as those along the lines, *CB* and *DE*, can produce bending moments only in a direction at right angles to the line of such shears, this fact should promptly be made known to the profession, as it would save a great deal of material in many forms of construction. It would be easy, for example, to arrange the end supports of a long slab in such a manner as to eliminate entirely the bending moments in either direction desired. Such a proposition does not appear to need any discussion.

Mr. Mensch believes that he has shown how a statical moment can be made to disappear, and has done so by showing how a thin slab supported mainly on transverse beams can have its bending moment in a transverse direction gradually reduced to zero by thickening the slab until it is equal in thickness to the beams. This is no doubt true, but a correct statical analysis, taking into account the supporting shears along the sides of the slab where it rests on the wall, would show it to be the case, and would require no mysterious explanation contrary to the laws of statics. Here, again, an error is made by not taking into account these shears.

In the case of a flat slab, the fact that it is supported at its four corners, instead of along its sides, appears to be a source of confusion. It may be of some assistance to those interested in the details of

* *Transactions*, Am. Soc. C. E., Vol. LXXXI.

Mr. Turneaure.

analysis to consider the problem in the following manner: Fig. 29 represents a slab in which the upper half is divided into strips one unit in width up to the point where the slab is supported on the columns. Each of these strips except the last one at the top has acting on it a downward load, consisting of the dead and live load, and shears along the sides. In the case of strip No. 1, the shear along the near side is zero, and the shear between No. 1 and No. 2 is equal to the load on strip No. 1. The distribution of this shear is not known, but if it were known, the moments in this strip could be accurately determined. If the shear were uniformly distributed, there would be no bending moment, but, obviously, it is of greater intensity near the ends than near the center, and the moment is positive at the center and negative at the ends. Proceeding further with the problem, it is obvious that strip No. 1 must transfer all its load to strip No. 2, and, likewise, strip No. 2 to No. 3, and so on, until we reach No. 7. As a consequence, there will be a shear acting between No. 6 and No. 7 the sum total of which is equal to the entire load on strips Nos. 1 to 6, inclusive. This shear acts downward on No. 7 and, taken together with the load on No. 7, is transferred to the columns on the lines, *CB* and *ON*, and to the column strip, *ABOG*. The total shear transferred from No. 7 along the line, *CN*, must be equal to the entire load on the half-panel between *CN* and the center, as there is no other place to which this load can go. This load amounts, approximately, to $\frac{W}{2}$. One-half of this is transferred to the columns, assuming the shears on *CB* and *ON* each equal to $\frac{W}{8}$; the other half, amounting to $\frac{W}{4}$ is transferred to the column strip, *ABOG*, and thence to the columns along *AB* and *GO*, there being no shear on *AG*. Therefore, it comes finally to this: that, although successive strips transfer their loads laterally up to the line of the columns, all of it must finally be carried to the columns on the shear lines, *CBA* and *GON*. If we wish to get the total bending moment in the slab on the lines, *JK*, *AB*, *CD*, and *EF* (positive plus negative), we may as well then consider the entire half panel to the left of *JK*, with column reactions of $\frac{W}{4}$ each. This will give a total moment for the panel of $\frac{1}{8}$ *W L*. The fact is that the advocates of

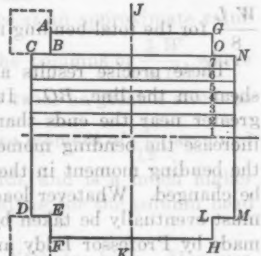


FIG. 29.

Mr. Turneaure. $\frac{1}{16}$ *W L* for the bending moment have, in effect, ignored the stresses in the column strips, *ABOG* and *EFHL*. They have simply obtained the total bending moment for a square slab, *BELO*, supported along its edges, *BE, EL*, etc. The shears on these edges are each equal approximately to $\frac{W}{4}$, and, assuming a uniform distribution of shear, the total bending moments in either direction will be $\frac{1}{16}$ *W L*; but, when we consider the column strip, *ABOG*, with its load applied along the line, *BO*, we will find a bending moment in this of $\frac{W\,L}{32}$. Adding the moment for the strip, *EFHL*, we get $\frac{W\,L}{16}$ for the two column strips, and $\frac{W\,L}{8}$ for the total bending moment.

These precise results are based on the assumption of a uniform shear on the line, *BO*. It would appear, however, that this shear is greater near the ends than along the center, which, in effect, would increase the bending moment in the square panel, *BELO*, and reduce the bending moment in the column strips. The total, however, cannot be changed. Whatever load is transferred laterally from strip to strip must eventually be taken by the column strips. A mistake apparently made by Professor Eddy and some others is in assuming that each of the strips can throw its load on its neighbor indefinitely. This appears to be the error in the analysis of Grashof, who, in his solution of a slab on point supports, derives a coefficient of substantially $\frac{1}{16}$. In this case the column strip, to which his loads are transferred laterally, disappears, but the moment of $\frac{1}{16}$ *W L* cannot be disposed of in this easy fashion. It still exists, but, acting on a strip of zero width, it produces stresses of infinity, a result of some interest mathematically, but of no practical value. Grashof in this way arrives at the anomalous result that all strips parallel to the *X*- or *Y*-axes are curved in exactly the same manner. This is obviously incorrect, as the distribution of shears and moments in the column strip, *ABOG*, supported at its ends, cannot possibly be the same as in the slab strips supported along their sides.

This situation seems to be clearly recognized by Professor Eddy himself.* He derives the same equations as Professor Grashof, but adds: "It is seen therefore that the form of solution which we are investigating implicitly assumes that at each edge of the panel there is

* In his book on the "Flat Plate Theory," p. 20.

some auxiliary form of structure that will bear the shears coming to it from each side, and at the same time assumes the curvatures and deflections contemplated in (21)." He further states that this solution results in a shear along each edge of one-quarter of the total panel load, and uniformly distributed along the edge. In the following pages of the book are discussed the action of the portion of the slab between columns as an auxiliary girder in which moments are produced by the above mentioned shears. This auxiliary girder is precisely the same as the column strips herein discussed, and in which the bending moment under the assumption of a uniform distribution of shear is $\frac{1}{16} W L$. It is difficult to reconcile the discussion of this point in Professor Eddy's book with his discussion of the Committee's report, especially with reference to the subject of directed shears. Mr. Turneaure.

It is interesting to note that Föppl* arrives at an approximate value for maximum fiber stress for the slab over the columns of $\frac{3 W}{\pi t^2}$, where $t =$ the thickness of the slab. This is nearly twice as great a stress as would be obtained from a coefficient for negative moment of $\frac{1}{12}$, assumed as uniformly distributed over the panel width, and is a much higher value than recommended by the Joint Committee for the column head sections. Föppl also adds:

"It may perhaps be that this estimate is a little too high. However, I scarcely believe that it is very far from the truth—a fact that I should like to emphasize particularly in view of another estimate which I consider much too low."

It seems quite likely that this statement refers to the Grashof analysis, made many years previously.

Professor Eddy seems to think that Professor Talbot and the writer differ as to the values of the shears. There is no conflict between Professor Talbot and the writer on this point, as a careful reading of the discussion will disclose. The writer agrees with Professor Eddy's general statement on page 1179,† with reference to shears, but this general principle is not followed at all in his later analysis on page 1184.† If it were, he would not have arrived at his results. Professor Talbot's lines, *AB* and *CD*, on page 1203† do not constitute perimeter lines, but overlap, and the shear along each one is correctly given as $\frac{W}{2}$, being made up of $\frac{W}{4}$ from the interior panel and twice $\frac{W}{8}$ for the narrow strips outside of the points of intersection.

* "Festigkeitslehre", 1914, p. 285.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI.

Mr. Turneaure. In conclusion, the writer insists that the analysis of the Committee is fundamentally sound, and based on sound principles, namely, the laws of statics. Those who wish to ignore those laws must resort to some form of guesswork, and one guess appears to be as good as another. The latest proposition relative to directed shears is a selective method, which is unsound and leads to absurd results when applied to the simplest problems.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1425

THE ACTIVITIES OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS DURING THE PAST TWENTY-FIVE YEARS*

By Chas. Warren Hunt, M. Am. Soc. C. E.

In 1897 a "Historical Sketch of the American Society of Civil Engineers" by the writer was published by the Society. This was issued in book form only, and a limited number sold, the proceeds being turned over to the Building Fund for the Fifty-seventh Street House. At the Washington Convention, in 1902 (the Fiftieth Anniversary of the Society), he briefly sketched the development of the intervening years. These, so far as known, form the only attempt at a connected account of the activities of the Society.

During the past quarter century many things have happened, and much has been accomplished of which there is no convenient and readily accessible record. It is true that much material, in a more or less fragmentary form, may be found scattered through the 250 monthly numbers of *Transactions* and *Proceedings* published during that period, but, even if they are all accessible in bound form, more effort and time are necessary to get at the facts than the busy engineer can afford.

In addition to this, the growth has been so rapid that only 646 (about 7½%) of the present membership of 8 544 were connected with the Society at the beginning of this period. It should be remembered also that the rate of increase in membership has been so much greater during the latter part of this period, that 5 137 (more than 65% of the increase) have joined within the last ten years.

* Presented at the meeting of December 5th, 1917.

With full recognition of the fact that statistical matter and figures are more useful in a printed than in a spoken record, it is intended to place before you this evening as briefly as possible the things which appear to be most interesting, and of which the membership in general has little if any information.

Early History.

The American Society of Civil Engineers was inaugurated at a meeting held in the office of the Croton Aqueduct Department, Rotunda Park, New York City, on Friday, November 5th, 1852. At this meeting 12 Engineers were present. Alfred W. Craven, Chief Engineer of the Croton Aqueduct, presided. The first Constitution (adopted December 1st, 1852) declared the object of the Society to be:

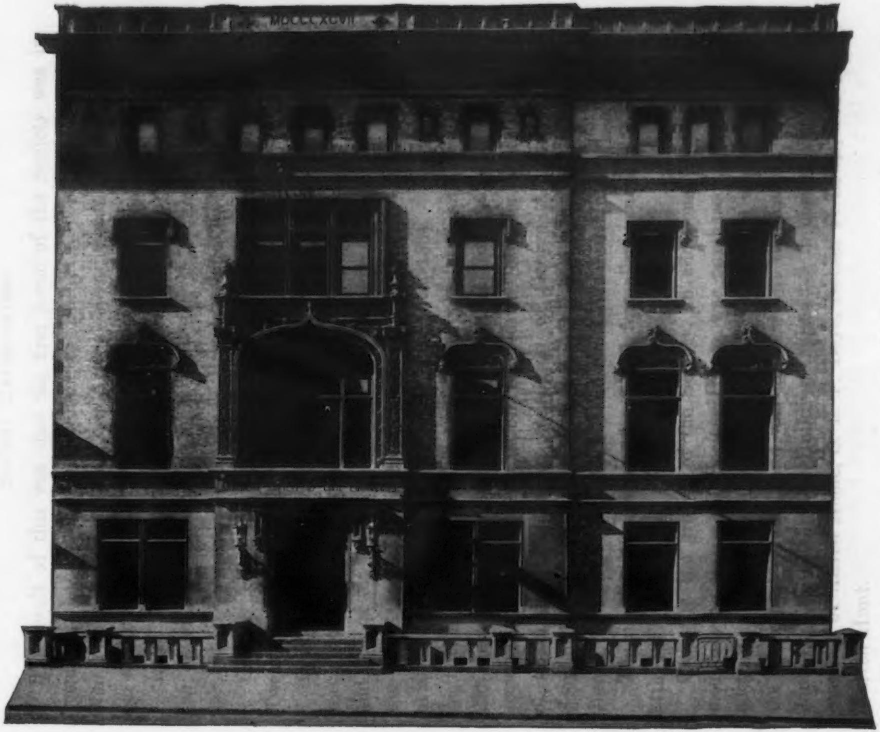
"The professional improvement of its members, the encouragement of social intercourse among men of practical science, the advancement of engineering in its several branches, and of architecture, and the establishment of a central point of reference and union for its members."

The circular issued at that time stated:

"Civil, geological, mining, and mechanical engineers, architects, and other persons who, by profession, are interested in the advancement of science, shall be eligible as members.

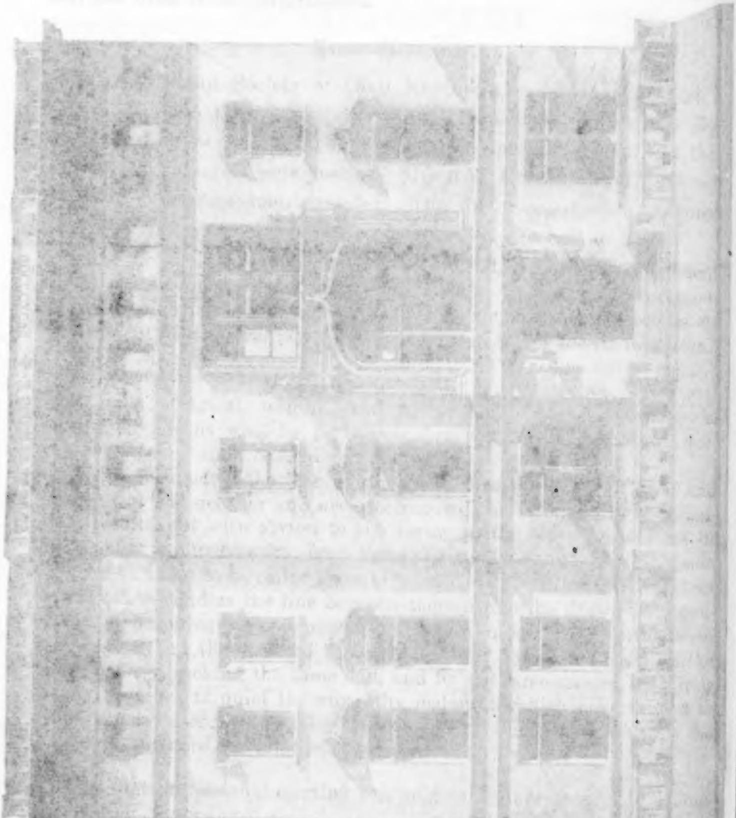
"It is anticipated that the union of the three branches of civil and mechanical engineering and architecture will be attended by the happiest results, not with a view to the fusion of the three professions in one; but as in our country, from necessity, a member of one profession is liable at times to be called upon to practice to a greater or less extent in the others, and as the line between them cannot be drawn with precision, it behooves each, if possible, to be grounded in the practice of the others; and the bond of union established by membership in the same Society, seeking the same end, and by the same means, will, it is hoped, do much to quiet the unworthy jealousies which have tended to diminish the usefulness of distinct societies formed heretofore by the several professions for their individual benefit."

The first professional meeting was held on January 5th, 1853. During 1853 and 1854, fourteen meetings, with an average attendance of six, were held, all in the office of the Croton Aqueduct Department. There is no record of any meeting after that of March 2d, 1855, at which the question of the securing of quarters was considered and the Society adjourned, until October 2d, 1867, when a meeting was held at the office of C. W. Copeland, 171 Broadway, New York City, at which the



HOUSE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

With full comparison of the data then recorded matter and figures are there needed in a printed [illegible] record, it is [illegible] to place before you this evening in brief [illegible] the things which appear to be most interesting, and of which the membership in general has little if any information.



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Minutes of the Meeting of March 2d, 1855, were accepted, and the object of the meeting stated to be "to take such steps as might be necessary to resuscitate the Society."

SOCIETY HEADQUARTERS.

The result of this was that the first home of the Society was in rooms in the Chamber of Commerce Building, 63 William Street, New York City, where the First Annual Meeting was held on November 6th, 1867.

In 1871 the quarters in William Street were enlarged by the renting of additional rooms, and on May 1st, 1875, new quarters were secured on the southeast corner of Broadway and Twenty-third Street.

On May 1st, 1877, the Society moved into a house, No. 104 East Twentieth Street, which it rented.

In April, 1881, a dwelling house, No. 127 East Twenty-third Street, was purchased, the first meeting being held there on May 4th, 1881, and it is of interest to note in passing that one of the Founder Societies—The American Institute of Electrical Engineers—came into being at a meeting held in that house on May 13th, 1884.

This house was occupied until 1896, when two lots, Nos. 218 and 220 West Fifty-seventh Street, with a total frontage of 50 ft., were acquired, and building operations started in December, 1896, in charge of a Building Committee consisting of George A. Just, Charles Sooysmith, Bernard R. Green, George H. Browne, William R. Hutton, Joseph M. Knap, T. C. Clarke, and Chas. Warren Hunt.

The new house was completed and formally opened on November 24th, 1897.

Owing to the growth of the Society, an additional 25-ft. lot, immediately adjoining the Society House, was purchased in 1904, and a 50% addition to the house was built. This addition was completed in the latter part of 1905, and was first used at the Annual Meeting of January 17th, 1906. The Building Committee in charge of this work consisted of Alfred Noble, S. L. F. Deyo, Nelson P. Lewis, and Chas. Warren Hunt.

The Society property then consisted of a plot of 75 ft. frontage on Fifty-seventh Street, varying in depth from about 107 ft. on the east, to about 117 ft. on the west. The House was a 4-story and basement, fireproof structure, the two lower floors covering the entire plot, and the two upper floors only the front portion. The first floor contained a

spacious foyer and three offices, one of which was used for the office of the Secretary. There was a large room in the rear called a Lounging Room, its use being principally for informal and social meetings. The main stairway gave access to the second floor on which there were in the front a large Reading Room, and in the rear an Auditorium with a seating capacity of 500. The third floor was devoted entirely to the office force, and the top floor to a double tier of book stacks with sufficient capacity for about 150 000 volumes, and with space for considerable enlargement. The building was a dignified and commodious one, and, having been specially designed for the use of the Society, proved itself adequate in every way, and, with certain additions which could have been made at any future time for the increase of space available for office and stack-room purposes, undoubtedly would have been ample for the use of the Society for many years to come. The total amount expended by the Society for the lots and building was, in round numbers, $360 000.

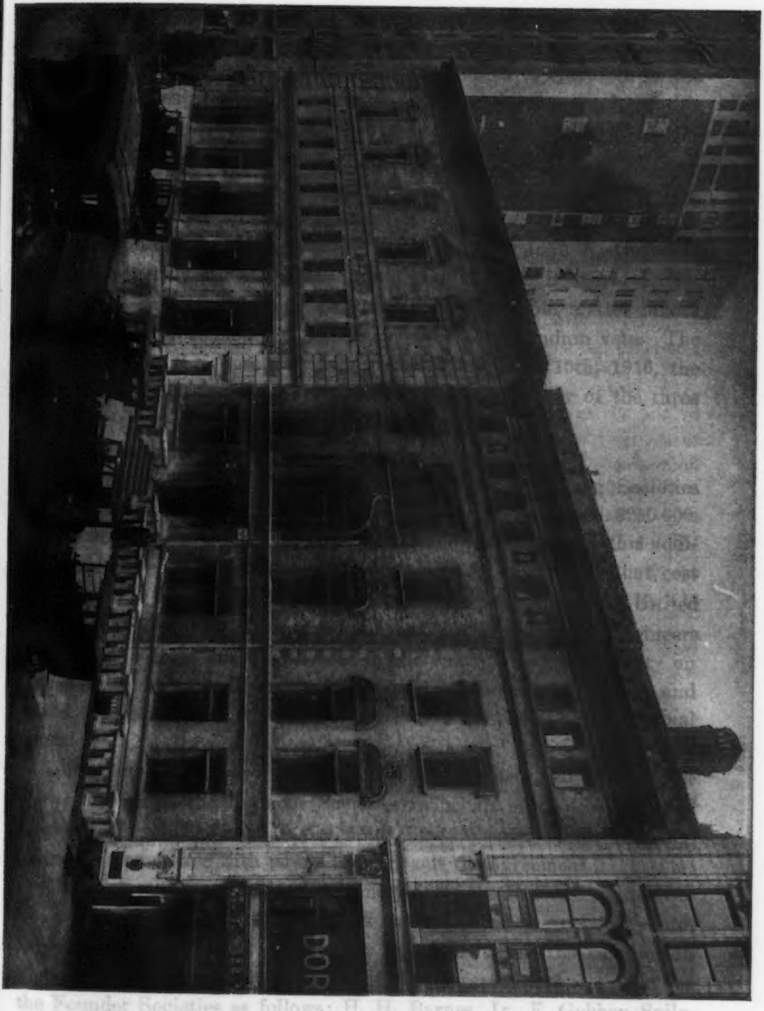
In February, 1903, Mr. Andrew Carnegie offered to give $1 000 000 to erect a suitable union building for the American Society of Civil Engineers, the American Society of Mechanical Engineers, the American Institute of Mining Engineers, the American Institute of Electrical Engineers, and the Engineers Club. This offer was very carefully considered by this Society, and submitted to a referendum vote of the entire Corporate Membership, the arguments for and against its acceptance being set out in an impartial manner. The result was that the membership decided, by a vote of 1 139 to 662, not to accept the offer.

The other organizations mentioned accepted. The amount donated by Mr. Carnegie was increased to $1 500 000, the result being the Engineering Societies Building, Nos. 29-33 West 39th Street, and the Engineers Club, 32 West 40th Street. The fund was divided as follows: to the three Engineering Societies, $1 050 000, to the Engineers Club, $450 000.

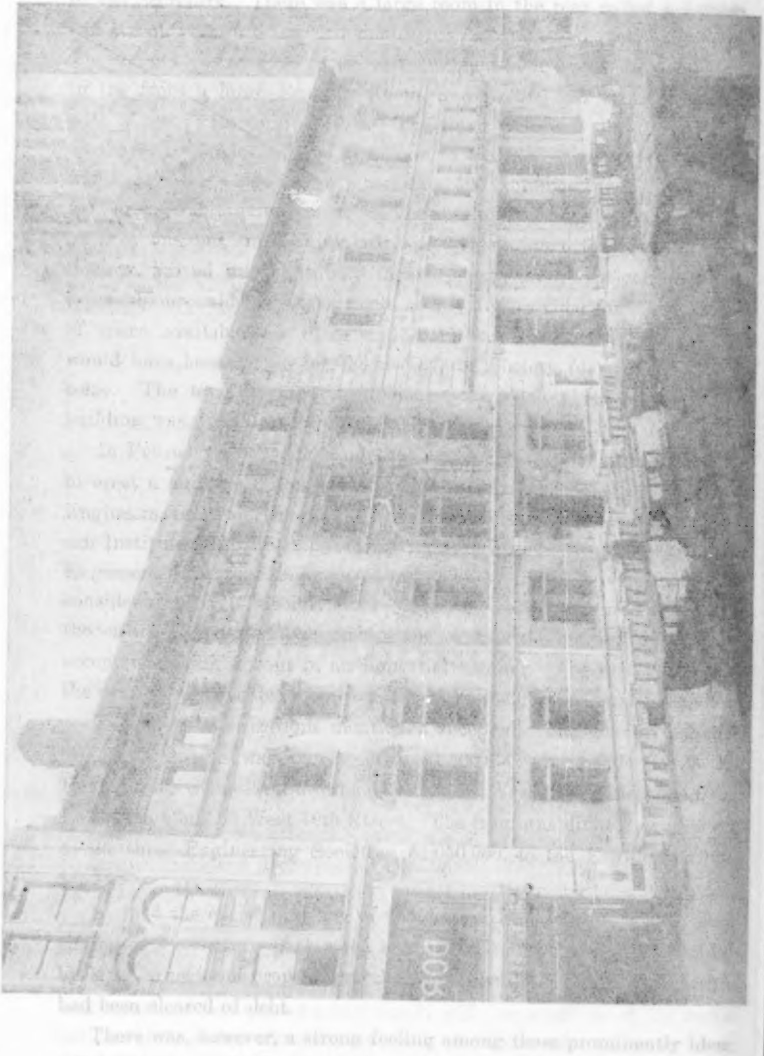
In 1914 the entire property of the United Engineering Society consisting of a structure of thirteen stories, built with the funds provided by Mr. Carnegie on property purchased by the three Founder Societies, had been cleared of debt.

There was, however, a strong feeling among those prominently identified with the activities of the three Founder Societies that this build-

ing could not be considered a strictly representative Professional Headquarters until it housed also the oldest of the National Societies



the Founder Societies as follows: H. H. Barnes, Jr., E. Gybbon Spilsbury, Chas. F. Rand, and Chas. Warren Hunt



ing could not be considered a strictly representative Professional Headquarters until it housed also the oldest of the National Societies.

After several preliminary discussions of the matter by individuals, on June 9th, 1915, an informal meeting of members of all the National Engineering Societies interested in the question of co-operation of the various branches of the Profession was held, and, as a result of this meeting, the matter was taken up by the Board of Direction of this Society, and Clemens Herschel, Robert Ridgway, and Chas. Warren Hunt, were appointed a Committee to consider the question of a possible amalgamation in an Engineering Headquarters. Charles F. Loweth, Hunter McDonald, George F. Swain, and John A. Ockerson were subsequently added to this Committee, and the Board of Direction, under date of February 1st, 1916, laid the whole matter before the Corporate Membership of the Society for a referendum vote. The letter-ballot on this question was canvassed on June 15th, 1916, the result being 2 500 in favor of the acceptance of the offer of the three Founder Societies to 390 against it.

This offer, briefly stated, was as follows:

That a three-story addition be made to the Engineering Societies Building at a cost estimated at $225 000, and not to exceed $250 000. That the American Society of Civil Engineers should pay for this addition, if the cost did not exceed the latter figure, but that if that cost exceeded $250 000 the additional expense should be borne by the United Engineering Society. That the American Society of Civil Engineers would then become an equal owner in the whole enlarged property on the same terms as each of the three original Founder Societies, and would occupy as much space as it might need on two of the additional floors.

Immediately afterward the Board of Direction accepted in due form the invitation of the Founder Societies in behalf of the Society, and Clemens Herschel, J. V. Davies, and Chas. Warren Hunt, were appointed a Committee with power to carry out the agreement.

This agreement was ratified at a meeting of the United Engineering Society on August 10th, 1916. Work was begun on the necessary preliminary structural work on August 1st, 1916, under the supervision of a Building Committee consisting of one representative from each of the Founder Societies as follows: H. H. Barnes, Jr., E. Gybbon Spilsbury, Chas. F. Rand, and Chas. Warren Hunt.

Owing to the general conditions of labor and material, the cost of the addition to the building, which it was thought in 1915 was amply provided for, with all contingencies taken care of, in the estimate of \$225 000, was found to be at least \$50 000 in excess of the limiting figure, or \$300 000. This additional cost has been borne equally by the four Founder Societies.

The total share of this Society, therefore, has been \$262 500, which, together with certain additional expenses in fitting up the new quarters, cost of new furniture, and moving, will bring the total expense of our change of headquarters to approximately \$280 000.

The addition, as before stated, consists of three stories. The fourteenth floor will be used as a stack-room for the United Engineering Library, headroom for a double tier of stacks having been provided. A report of the writer to the Board describes our new quarters, as follows:

"The lay-out of the floors to be occupied by this Society was made by the writer with a view to utilizing every available foot of space and to secure good light. This was the more necessary inasmuch as the floor area of these two floors is much less than that of the lower floors.

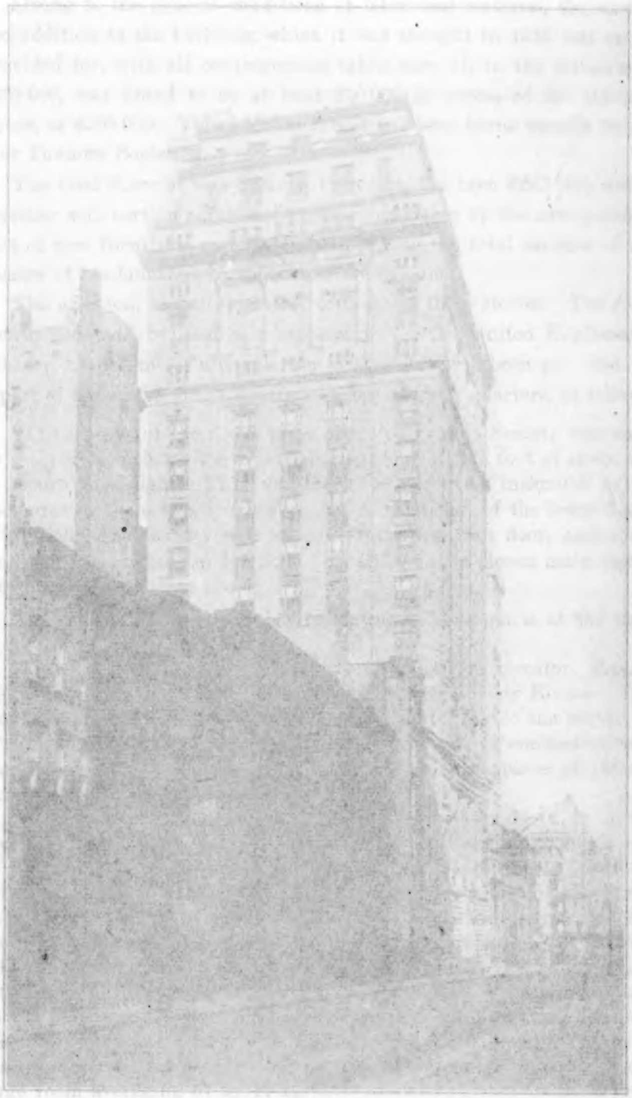
"Briefly, the Society will occupy the entire 15th floor, and about two-thirds of the 16th or top floor. In all there are eleven main rooms. On the 15th floor there are:

"(1) The office of the Secretary, entrance to which is at the right of the elevators.

"(2) The Reading Room, directly opposite the elevator, the entrance to which will be the main entrance to the Society Rooms. This room is 51 by 26 ft. and looks out over Bryant Park to the north. It is panelled in oak, and when used by our members, in connection with the Library will, it is believed, practically take the place of the old Reading Room in Fifty-seventh Street.

"(3) The Board Room. This room, which is 43 by 24 ft., is on the south side of the building, directly opposite the Reading Room, a 6-ft. hallway separating them. This room is panelled in mahogany, and the furniture for it, which has been specially designed, is also of mahogany, and consists of 4 tables and 30 chairs. The tables are designed so that they can be placed together, making a table 24 by 6 ft., or can be separated and used as units 6 by 6 ft.; and, when necessary, can be made into tables 6 by 3 ft. to set against the wall and take up very little room. In the partitions between these rooms and the hallway, two 8-ft. openings, opposite each other, with sliding doors, have been arranged, so that the two rooms can be thrown together, practically forming one large room averaging 57 by 47 ft.

ENGINEERING SOCIETIES BUILDING
33 WEST THIRTY-NINTH STREET



ENGINEERING SOCIETIES BUILDING
33 WEST THIRTY-NINTH STREET

"(4) General Office. A large room covering the east side of the building, 59 by 37 ft. Here will be located the general office force. A service stairway, which will practically be a private stairs for this Society, gives access to the 16th floor, where, on the east side of the building, there are four small offices, one of which (5) is to be used as a Rest Room for women; (6) for the Bookkeeper; (7) Editorial Department; (8) Applications Department. Three other large rooms are available for Committee Rooms, or whatever use may develop in the future. They are (9) 24 by 20 ft., (10) 22 by 24 ft., (11) 36 by 23 ft.—these figures being approximate.

"A doorway in the hall separates that part of the 16th floor to be used by the Society from three rooms which are available for renting by the United Engineering Society, and to which access is obtained through the elevator and hallway without passing through the quarters of the Society."

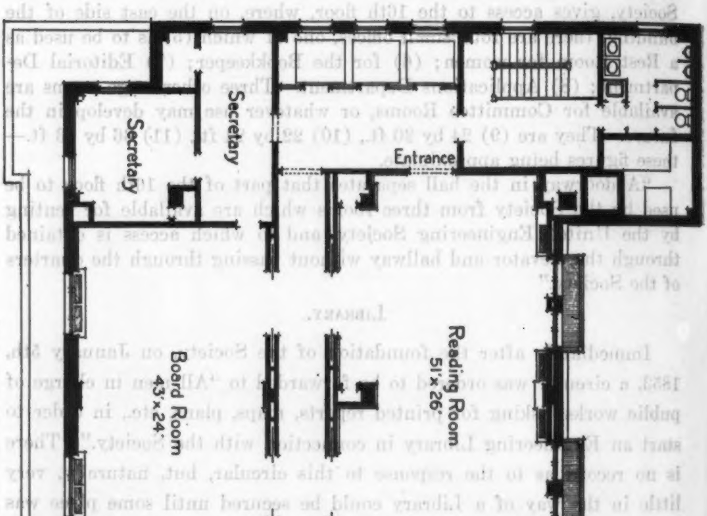
LIBRARY.

Immediately after the foundation of the Society, on January 5th, 1853, a circular was ordered to be forwarded to "All men in charge of public works, asking for printed reports, maps, plans, etc., in order to start an Engineering Library in connection with the Society." There is no record as to the response to this circular, but, naturally, very little in the way of a Library could be secured until some place was provided in which the books could be cared for, and it was not until headquarters were first established, in 1867, that the Library really had a start. After that its growth was quite rapid, when one considers that practically no books were purchased, the accessions being entirely the result of donations. Several large additions were received in the succeeding years, notably, in 1872, one from William Young Arthur, M. Am. Soc. C. E., and in 1873 one from William J. McAlpine, Past-President, Am. Soc. C. E.

The Annual Report of the Board of Direction for 1873 gave the total contents of the Library as 3 433.

In 1873 a special committee was appointed, under the following resolution:

"Whereas, the foundation of a library and museum, which contains within itself all accessible published matter relating to the history, theory and practice of engineering, the construction and management of public improvements, and the methods and cost of manufacturing operations, with illustrations by models and samples of the results thereby obtained, must be invaluable, not only to the profession, but



AMERICAN SOCIETY OF CIVIL ENGINEERS
ENGINEERING SOCIETIES BUILDING, FIFTEENTH FLOOR

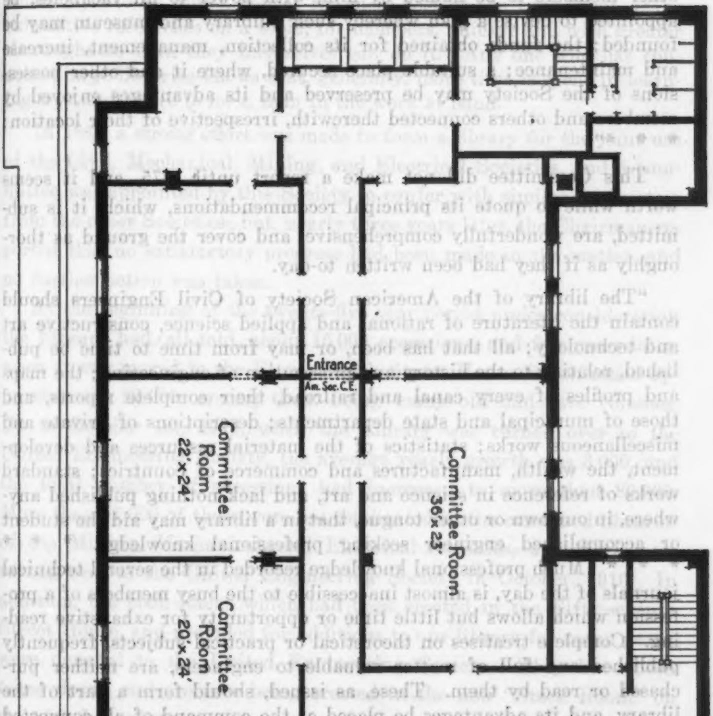
to all who are interested in the pursuit or the application of practical knowledge,

"Resolved, that a Committee, consisting of the President and nine other members to be named by him, with power to fill vacancies, be appointed to prepare a plan by which such library and museum may be founded; the means obtained for its collection, management, increase and maintenance; a suitable place secured, where it and other possessions of the Society may be preserved and its advantages enjoyed by members and others connected therewith, irrespective of their location. * * *"

This Committee did not make a report until 1875, and it seems worth while to quote its principal recommendations, which, it is submitted, are wonderfully comprehensive and cover the ground as thoroughly as if they had been written to-day.

"The library of the American Society of Civil Engineers should contain the literature of rational and applied science, constructive art and technology; all that has been, or may from time to time be published, relating to the History and Practice of engineering; the maps and profiles of every canal and railroad, their complete reports, and those of municipal and state departments; descriptions of private and miscellaneous works; statistics of the material resources and development, the wealth, manufactures and commerce of countries; standard works of reference in science and art, and lack nothing published anywhere, in our own or other tongues, that in a library may aid the student or accomplished engineer seeking professional knowledge. * * *

* * * Much professional knowledge recorded in the several technical journals of the day, is almost inaccessible to the busy members of a profession which allows but little time or opportunity for exhaustive reading. Complete treatises on theoretical or practical subjects, frequently published, and full of matter valuable to engineers, are neither purchased or read by them. These, as issued, should form a part of the library, and its advantages be placed at the command of all connected therewith, wherever they may happen to reside, so that at their request, complete examinations on specified topics can be made, pertinent extracts copied, and proper references given.



AMERICAN SOCIETY OF CIVIL ENGINEERS
ENGINEERING SOCIETIES BUILDING, SIXTEENTH FLOOR

"The plan here outlined involves the preparation of concise abstracts of new works, reports, scientific and technical journals, proceedings of societies, and other publications, as received; the whole to be classified and indexed, that a busy man may quickly learn, without the trouble and expense of looking over the vast amount of matter now published, to determine for himself, whether there has recently appeared in print anything referring to a particular subject. A serial index of current engineering and technical literature as thus described, can be comprised within a few pages issued weekly or monthly, and

to all who are interested in the pursuit or the application of practical knowledge,

"Resolved, that a Committee, consisting of the President and nine other members to be named by him, with power to fill vacancies, be appointed to devise a plan whereby such a library and museum may be founded; the funds obtained for its collection, management, increase and maintenance; a suitable place secured, where it and other possessions of the Society may be preserved and its advantages enjoyed by members and others connected therewith, irrespective of their location; * * *"

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would largely facilitate the dissemination of professional knowledge 'among men of practical science'.

"A skillful librarian, who knows what the library contains, and where it is to be found, can at the mere cost of the time spent, make exhaustive researches on a topic, for members, quicker and with greater thoroughness than they themselves can do it. Any one who has consulted large libraries knows that, generally, more time is spent in learning how and where to look, than in the work at hand."

In 1885, a strong effort was made to form a library for the joint use of the Civil, Mechanical, Mining, and Electrical Societies, and a committee was appointed by this Society to confer with similar committees from the other Societies; but, nearly three years later, the Chairman reported that no satisfactory progress had been made in the matter, and no further action was taken.

At the beginning of the twenty-five year period under consideration the Library had, all told, about 16 000 accessions, and five years later. when it was moved to the Fifty-seventh Street House, it contained approximately 22 000, among them being many old and rare volumes. Up to October 1st, 1916, when the Library was turned over to the United Engineering Society, the average yearly growth was 3 000, and the total number of accessions had increased to more than 89 000. More than 67 000 of these were not duplicated in the combined libraries of the Mining, Mechanical, and Electrical Societies, and these were turned over to the United Engineering Society in October, 1916. In addition, the book-stacks which had been erected in the Fifty-seventh Street House, and provided for additions to our library for many years, were donated to the United Engineering Society. They have been taken down, and are now being erected in the new "Stack Room" on the 14th floor of our new home.

The remaining 22 000 volumes have been presented to the Cleveland Association of Members. The collection is to be kept intact, and is now temporarily in the custody of the Cleveland Public Library.

In the Fifty-seventh Street House provision had been made for a commodious, up-to-date Stack Room, and, immediately upon moving in, a thorough re-classification and indexing of the Library was undertaken. The Library at that time was in an exceedingly chaotic state. No systematic index for it had ever been made, and it was a problem how it should be made efficient and available for the use of Engineers. The task fell upon the writer, and he made every effort to find out just

what had been done up to that date in the classification and cataloguing of an Engineering Library, by inquiry from available sources. A composite picture of the replies received would have read somewhat like this: "We use such and such a system, and we advise you not to." Under this condition he was thrown entirely on his own resources, and the classification which has been in use for 20 years (it is still used so far as our books, which have been transferred to the United Engineering Library, are concerned), was worked out.

In such a pioneer effort by one who, up to that time, had a very limited knowledge of Library work, it is not surprising that there were many imperfections. On the other hand, it was put together from the standpoint of an Engineer, and experience has shown that it has been a most efficient tool. This classification was used, not only to arrange books on the shelves, but also to arrange cards in the Catalogue. Many of the classes were very large, and were not sub-divided closely, and therefore the "Class Catalogue" was supplemented by a "Subject Catalogue" in which the cards were arranged alphabetically by subject. At least one card was written for every book in the Class Catalogue, and as many additional cards were placed in either the Class or Subject Catalogue as were necessary to cover its contents fully. All books were very carefully analyzed, cards being written for any sections or chapters which would be of special interest, which necessitated in some cases as many as 40 or 50 cards for one book. In addition to the two Catalogues described, there was also an "Author Catalogue" in which at least one card was filed for every book in the Library.

In 1900 the Classified Catalogue was printed and issued in a volume to all members. This book contained 700 pages, and covered about 32 000 titles. Its issue stimulated the growth of the Library to such an extent that two years later a second volume of 293 pages was issued, bringing it up to date.

During the years in which this classification was in use much experience was gained, and toward the latter part of that period an improved and extended classification was worked out by two members of the Library Staff, Miss Eleanor H. Frick, and Miss Esther Raymond, on their own initiative, and largely in their own time.* Though this classification is based on the general ideas of the writer, full credit for the work belongs to the Librarians mentioned. It is believed that the

* The two classifications are given in Appendices A and B.

publication of these two classifications will be of considerable use, not only to Technical Libraries, but to members of the Profession. As an instance of such use, it may be stated that the Committee of Engineering Council charged with tabulating the members of the Society available for special work in connection with the War, used this classification in making up the various headings under which the members of this Society should be indexed.

In 1896, the writer, in the "History" previously referred to, speaking of the Library, said:

"While it is not possible now to bring its use within the reach of members residing at a distance, it is hoped and believed that after the new house is completed arrangements can be made by which non-resident members may be able to secure data on any special points at small expense."

As soon as possible after the cataloguing had been completed, he took up the matter, and in 1902 was authorized by the Board to make searches in the Library, upon request, and to charge therefor the actual cost to the Society of the work required. About 1 000 such searches and bibliographies have been gotten out, and there is abundant evidence of the appreciation of our non-resident membership.

A number of years after this system was started, the Library of the United Engineering Society established its Service Bureau, which has been very successful; and, as our Library now forms part of the consolidation, our members will have the benefit of that service.

Local Associations.

The question of the formation of Local Associations of Members in the various centers of population was considered in a general and informal way several times prior to 1905. It was discussed at the Cleveland Convention in that year, following a report from the Secretary stating that a circular note had been forwarded to at least three Members in each of the following cities: Albany, Boston, Cleveland, Chicago, Detroit, Kansas City, Mexico, New Orleans, Philadelphia, Pittsburgh, St. Louis, St. Paul and Minneapolis, San Francisco, and Washington, setting forth the advantages of such Associations, both locally and to the Society as a whole, recommending their formation, and enclosing a draft of a proposed Constitution suitable for adoption. The Secretary reported that considerable interest had been

aroused, and that two Local Associations had been formed, one at Kansas City, Mo., and one at San Francisco, Cal.; that meetings had been held at Washington, Cleveland, Pittsburgh, Boston, St. Louis, and Philadelphia, and that a report from the three Chicago Members had also been received. The reports from Washington, Cleveland, and Pittsburgh, were non-committal. In Boston it was the unanimous opinion of those consulted that it would be very difficult to arouse sufficient enthusiasm; in St. Louis a meeting of 23 Members adopted a resolution to the effect that it was not desirable at that time to have such an organization in that city. In Philadelphia a letter-ballot was taken resulting in a vote of 42 to 14 against the proposition, and the Committee in Chicago was strongly against it.

The general idea of the organization of Local Associations of the Society, suggested by the Board of Direction, was approved by the Convention.

The writer remembers well what a hard struggle it was to overcome the many objections raised, the principal one being the fear that such Associations would injure local societies and clubs already established; but time has accomplished what then seemed impossible, and we now have Local Associations in each of the cities named except Albany, Boston, Mexico, Pittsburgh, and Kansas City. In the last named the first association was formed, but it was not successful. In addition there are 13 others, a total of 21. It is undoubtedly a fact that these Associations add strength to the Society as a whole, and are of great local benefit. Since the above was written, the writer has been informed unofficially of the formation of an Association in Pittsburgh.

An important meeting of the presidents of all the Local Associations was held at the Society House on January 19th, 1915, at which many matters of vital interest to the Society were discussed.

MEMBERSHIP.

Twenty-five years ago the total membership of the Society was 1 609; at the present writing it is 8 544, a net increase for that period of 6 935, the average yearly net increase having been 277. It should be noted that this increase has been in spite of the fact that the requirements have been raised during the period. The writer's opinion is that it is also due to this fact.

FINANCES.

As nearly as can be determined, the cash value of the property of the Society, at the beginning of the twenty-five year period under consideration, was $60 000. In a statement issued by the Board of Direction in May, 1895, when the building of the Fifty-seventh Street House was first contemplated, the available assets of the Society were given as follows:

House, 127 East 23d Street (estimate)	$60 000	
Mortgage	16 000	$44 000
Securities in safe deposit, par value		16 000
Cash, awaiting permanent investment		4 500
Amount available		$64 500

At the present time a similar statement would read about as follows:

Society House, 220 West 57th Street, cost	$360 000	
Less Mortgage	150 000	$210 000
New 39th Street Quarters, cost to the Society		267 500
Securities in safe deposit		10 000
		$487 500

The assets of the Society on the basis of this statement have increased during the past quarter century about $425 000. This, however, is very conservative, inasmuch as in the above figures the cost of the Fifty-seventh Street property is used, whereas in the statement of 1895 the value of the Twenty-third Street house was estimated, and largely in excess of the price paid for it; in addition to this, the value of the Society's one-fourth interest in the Thirty-ninth Street property is at least $250 000 more than the cost given. It would be more nearly correct, therefore, to say that the increase of property assets during this period has been $700 000.

Meetings.

During the past twenty-five years about 500 regular meetings of the Society have been held. Nearly all of these have been for the purpose of presenting and discussing professional papers and topics, and there have been 20 or 30 extra or special meetings, and about 50 meetings which are spoken of in the Constitution as for "social" purposes. There were also a number of special meetings of the Juniors of the Society.

Among the most notable events, the following might be mentioned:

The formal opening of the Fifty-seventh Street House on November 24th, 1897, was held in the afternoon. The President, Benjamin M. Harrod, of New Orleans, La., presided. The ceremonies were opened with a dedicatory prayer by the Rt. Rev. Henry C. Potter, and addresses were made by Gen. W. P. Craighill, Past-President, Am. Soc. C. E., J. G. Schurman, LL.D., President of Cornell University, and the Hon. Joseph H. Choate.

On September 16th, 1904, a reception was given to the members of The Institution of Civil Engineers of Great Britain, who were visiting this country by invitation of the Society.

On November 30th, 1910, at the home of the Society, the John Fritz Medal was awarded to the late Alfred Noble, Past-President, Am. Soc. C. E.

On June 3d, 1912, the Society tendered a reception to the Twelfth International Navigation Congress, and on September 5th of the same year to the members of the Sixth Congress of the International Association for Testing Materials.

From 1903 to 1910 all the meetings of the John Fritz Medal Board of Award were held in the Society House, and on many occasions meetings of other societies and associations were held there by special permission of the Board of Direction.

Amendments to the Constitution.*

A revised Constitution was adopted on March 4th, 1891, the principal changes being the provision for two new grades of membership. The class of Associate Member was created, so that it would be practicable to raise the qualifications for the highest grade, and to take

* All the amendments, with a brief statement of their purport and the vote by which they were adopted or rejected, will be found in Appendix C.

care adequately of a certain class of engineers not eligible for the grade of Member, as well as to provide at the proper time a method for advancement to Corporate Membership of those in the old Junior grade who were deserving of such advancement. The requirements for the grade of Junior were lowered so as to bring them within the reach of all young men who at the beginning of their careers wished to be connected with this Society. Provision was also made for an increase in the number of Vice-Presidents and for the enlargement of the Board of Direction, so as to make it more truly representative. The respective terms of office were lengthened, and it was stipulated that members of the Board should not be eligible for immediate re-election, thus securing rotation in office.

The Report of a Committee on Revision of the Constitution, under date of November 5th, 1890, signed by W. P. Shinn, Mendes Cohen, F. Collingwood, and S. Whinery, states in part:

"It was upon the question of the duties, position and standing of the Secretary that the greatest diversity of views was found to exist. A large number of members have expressed the opinion that the Secretary of the Society, like the secretary of an ordinary business corporation, should be appointed by the Board of Direction, but those who so think forget or ignore the fact that, unlike the ordinary business corporation, the offices of President and Vice-President in this Society are of an honorary nature. The homes of these officers are most frequently in parts of the country remote from the Society's place of business, and it may often occur that they can perform but few of the executive duties. In fact the Society does not contemplate that the men whom it honers with such positions shall drop their professional duties to attend to Society work, and it certainly does not propose to pay them for doing so. The executive duties must, however, be performed by some one, and at all times. The Committee has, therefore, distinctly named the Secretary, under the President and Board of Direction, the executive officer of the Society.

"If we stop for a moment to consider the important duties to be performed by such officer, often of a delicate and confidential character, it will be seen that he should have a voice in the deliberations of the Board; for he is the source of all information, and to him must be referred the detailed investigation of every question.

"It is necessary, too, that the office should be filled by a person capable of representing the Society favorably, and deciding properly in the matters constantly arising in the intervals between the meetings of the Board; and this can only be well done by a professional man, of business experience and standing. Such a man cannot be easily

secured for any sum which the Society can at present afford to pay; nor would such a man be willing to sever himself entirely from the field of professional engagement."

Up to 1894 the office of Secretary had been filled by a general vote of the membership, but in that year an amendment was carried placing the election of the Secretary in the hands of the Board of Direction, but otherwise not changing his status. The vote on this amendment was 191 to 6. In 1895 an amendment was carried which divided the territory occupied by the Society into 7 Geographical Districts and provided for representation of each of these Districts on the Board of Direction. The vote on this ballot was 273 to 12.

The revised Constitution adopted in 1891 provided for the election of all members by a letter-ballot of all Corporate Members, 7 negative votes excluding. It also provided that the Board, upon receipt of eight requests for reconsideration of the ballot in the case of any rejected candidate, was empowered to order another ballot to be taken. On this "Reconsideration" negative ballots to the number of 10% of the votes cast were necessary for exclusion.

The small number of negative ballots necessary for exclusion on the first ballot caused trouble by the exclusion of well-qualified applicants; the reconsideration ballot also proved unsatisfactory, for the reason that the number of ballots necessary for exclusion was dependent on an unknown quantity. Under it, a candidate might be excluded with only 15 negative ballots, and another might be admitted with 40 or more negative ballots. In fact, such cases as these actually occurred.

In 1903, the number of negative ballots required for exclusion on the first ballot was increased from 7 to 20. Even this proved unsatisfactory, and in 1908 the Constitution was amended by transferring the election of members of all grades from the membership at large to the Board of Direction. The vote on this amendment was 892 to 317.

In 1915 in order to provide for a more general representation on the Board of Direction, the territory occupied by the Society was divided into 13, instead of 7, Districts, each to be represented on the Board of Direction, the vote on this question being 1 066 to 83.

A number of amendments to the Constitution have been proposed and rejected. Among the most important of these was one, submitted in March, 1907, increasing certain of the admission requirements, particularly for the grade of Member. This was lost by a vote of 429 to 847.

In 1914 an amendment was offered which would have changed the status of the Secretary of the Society by excluding him from membership on the Board of Direction. This amendment was lost by a vote of 1 343 to 1 828.

ENGINEERING CONGRESSES.

Three International Engineering Congresses in which the Society was active, have been held in the United States. The first was held in 1893 in connection with the World's Columbian Exposition at Chicago. This Society took charge of Division "A", Civil Engineering, the work of which was described at the joint meeting of all divisions, August 5th, 1893, as follows:

"Six sessions have been held, and the work accomplished can be best shown by the following statement: Sixty-three papers in all were presented. Of these fifty had been printed and distributed for discussion, and covered about 1 200 pages of printed matter, with numerous plates and cuts.

"The subjects treated may be classified under the following heads:

"Common Roads; Railways, Terminal Systems, Signaling, Locomotives, etc.; Cable Railways; Bridges, Substructure and Superstructure; Canals; Foundations; Surveys and Surveying Instruments; Metals—Their Treatment for Substructural Purposes; Grain Elevators; Paving Brick; Carbon—Its Use in Electrical Engineering; Electric Light Plant; Hoisting Machinery; Inland Transportation; Navigation Works; Improvement of Rivers; Improvement of Harbors; The Plant of Commercial Ports; The Laying Out of Cities; Water Works; Sewers and Sewerage; Tunnels, and The Testing of Building Material.

"Twelve countries are represented in the authorship of these papers, as follows:

Germany furnished	20	Canada	3
Mexico	6	Italy	1
Portugal	5	Australia	1
England	3	United States	18
Holland	2		
France	2	Making a total of	63
South America	2		

"The work of translation of papers presented in foreign languages has been done in every instance by volunteers from the membership of the Society, by gentlemen thoroughly conversant with the subject under consideration.

"The interest manifested in the papers presented is evidenced by the fact that 318 engineers registered during the session of this Division, and the average attendance at each session was about 125.

"The discussions have taken a wide range, and, on account of the limited time, have been entirely confined to those presented orally. Many interesting and valuable written discussions were received, which it was entirely impossible to present at the sessions, but which will be published in connection with the papers.

"The number of valuable additions to the literature on the subjects mentioned is so great that it is impossible in this summary to do them all justice, and it is thought best not to attempt it.

"It may, however, be asserted that the results of the sessions of this Division of the Congress will be far-reaching and productive of great benefit to the profession of Civil Engineering all over the world."

The second International Engineering Congress was held in connection with the Louisiana Purchase Exposition at St. Louis, Mo., in October, 1904.

In 1903 this Society was invited by the Directors of the Louisiana Purchase Exposition to undertake the arrangements for an International Engineering Congress. Our Board of Direction appointed a Committee, and this Committee invited the co-operation of the other National Engineering Societies, but, for some reason which was never explained, they did not entertain the proposition favorably. Inasmuch as the inauguration and conduct of the proposed Congress had been placed upon this Society by the management of the Exposition, the Board determined, on January 4th, 1904, that the Society should undertake it alone, assuming the entire cost.

At that date nothing, even of a preliminary nature, had been done, and the organization, the securing, editing, and publishing of papers and discussions, as well as arrangements for meetings, devolved entirely upon the writer and his staff.

The first paper was received on March 29th, 1904, and between that date and October 1st, 1904, 83 papers were edited, printed, and circulated in advance, many discussions being received. The work of translating many of these foreign papers was undertaken by volunteers from the membership of the Society.

The Congress was held from October 3d to 8th, 1904. Its activities were divided into eight sections, 28 meetings were held, the average attendance at each being 50. In the discussion of the 38 selected sub-

jects, 97 formal papers, written by prominent specialists by invitation, were presented. In addition, 78 communications from engineers unable to be present were read, and there were 272 oral discussions at the Sectional meetings.

The proceedings were published subsequently in six extra volumes of *Transactions*, every member of the Society receiving copies of these volumes free of charge. The total edition was 4 000, and, in addition, separate pamphlets covering each of the subjects were printed, a total of 43 575 separate pieces being handled.

From foreign sources 46 out of a total of 96 papers, and 91 out of a total of 302 discussions, were furnished.

The attendance at the Congress was: from the United States 724; Canada, Cuba and Mexico 17; South America 10; Europe (13 countries) 111; Asia 10; Australia 4; a total of 876.

The total cost was $38 500, of which about $5 000 was received from subscription and sales of publications, the total net cost met by the Society being about $33 500.

The third International Engineering Congress in which the Society participated was held in connection with the Panama-Pacific Exposition, in San Francisco, Cal., September 20th-25th, 1915.

The plan of management of this Congress and the method of financing it, both of which were suggested by the writer, were as follows:

The original financial plan was that the cost should be underwritten as follows:

(1) By a general subscription from engineers residing in the Pacific Coast region........		$10 000
(2) By the five National Societies, in the following proportion:		
American Society of Civil Engineers.......	$9 000	
American Institute of Electrical Engineers..	9 000	
American Society of Mechanical Engineers..	5 000	
American Institute of Mining Engineers....	5 000	
Society of Naval Architects and Marine Engineers	2 000	$30 000
The estimated cost of the Congress was..............		$40 000

A General Committee of Management was composed of the President and Secretary of each of the four Founder Societies and of the

Society of Naval Architects and Marine Engineers, with four additional members from each Society resident in San Francisco.

The ten officers of the Societies mentioned formed a Committee on Participation, through which invitations to take part were transmitted to other Engineering organizations both at home and abroad. This Committee also arranged for providing the funds necessary to carry on the work.

The members of the Committee resident in San Francisco formed a Committee of Management to carry out the work in detail on the ground, W. F. Durand being Chairman and W. A. Cattell, Secretary-Treasurer.

This Committee took charge of the receipt, editing, printing, and distribution of the papers and discussions, which were finally issued in 13 volumes.

The total cost of the Congress was approximately \$77 000. Of this amount:

Pacific Coast Engineers contributed................	\$10 413.00
American Society of Civil Engineers contributed......	7 740.00
American Institute of Mining Engineers contributed..	4 300.00
American Society of Mechanical Engineers contributed.	4 300.00
American Institute of Electrical Engineers contributed.	4 300.00
Society of Naval Architects and Marine Engineers contributed....................................	1 720.00
Total..	\$32 773.00

The remainder of the total expense was received from membership fees, sale of additional volumes, etc., etc.

The Annual Convention of this Society was held in San Francisco during the week before the Congress, and similar meetings of the other Founder Societies were also held, thus assuring a good attendance. This was a somewhat memorable occasion, inasmuch as a special transcontinental train for the accommodation of the members of all these organizations, and other members of the Congress, was arranged for by the Joint Committee on Entertainment and Transportation of which the writer was Secretary.

The Congress consisted of opening and closing sessions, and 51 technical meetings. The total attendance was approximately 800, and

there were about 50 official delegates. Owing to the state of war existing in Europe, the foreign participation was much more limited than had been expected when the Congress was originally undertaken.

The product of this Congress was not distributed gratis to any of the members of the Societies participating, as was the case in 1904.

PUBLICATIONS.

The first paper printed by the Society was an Address delivered by President James P. Kirkwood directly after the reorganization of the Society in 1867.

The number of *Transactions* for November, 1873, was the first issued. The first 57 papers, which were printed separately, make up Volume 1 and part of Volume 2. Volume 3 begins with the number of *Transactions* for May, 1874, and Volume 4 with that of April, 1875. Between that date and 1886 the number of pages published was only sufficient to fill one volume per annum, but, beginning with 1887, and continuing until 1892, two were issued yearly, the total number of volumes up to that date being 28. In 1893 two extra volumes of *Transactions* were issued containing the product of the Civil Engineering Section of the International Engineering Congress.

Up to the end of 1895 the *Proceedings* and *Transactions* were issued together in monthly numbers, and, in order to preserve them for future reference, they had to be separated and bound in individual volumes.

The difficulty with this method was that a paper intended to be submitted to the Society was not published until it had been read at a meeting, and the discussion upon it, which was limited to the few who attended the meeting or who had received advance copies, had been edited, printed, and collated. Under these conditions the membership of the Society at large never saw or heard of any paper until the discussion of it was complete, which frequently was six months, and in some cases as long as eighteen months, after the paper had been received. The result of this was that the monthly numbers of *Transactions* lacked current interest, and when received by members frequently remained in their wrappers until sent to the binder when the entire yearly volume had been received.

The writer well remembers that one of the first pieces of work assigned to him as Assistant Secretary, in March, 1892, was the getting ready for publication of the number of *Transactions* for September of the preceding year.

In 1892-95 the issue, in addition to the regular *Transactions*, of a *Bulletin* in leaflet form, calling attention to current events and giving abstracts of the papers in advance of the date at which they were to be presented, was tried. The great difficulty with this was the preparation of proper abstracts. The experience of the writer leads him to the belief that a technical abstract, in order to be really good, must be prepared by one who is expert in the particular subject treated, and that, even in this case, he must study the paper carefully and write the abstract in his own words. Any attempt to produce an abstract of a paper by quoting here and there a paragraph is not productive of satisfactory results.

In January, 1896, the publication of our present monthly *Proceedings* was begun, the technical matter contained in these being subsequently collated and published in volumes of *Transactions*.

This method was new in Society publications, and has since been adopted by others. By it the member is interested in the receipt of his monthly Number, because it contains: (1) brief accounts of Society business, including abstracts of minutes of Society Meetings both in New York and in the headquarters of Local Associations, list of additions to the membership, announcements of future meetings, and other items of general interest; (2) not only the papers to be presented, but also the discussions upon them, which are published serially until each subject is exhausted.

It is a matter of pride that, during the 22 years that this publication has been issued, it has never failed to be mailed to the membership on the fourth Wednesday of the month, although at times the issues have contained as much matter as an ordinary volume, in one case 650 pages.

In March, 1899, the writer was authorized by the Board to publish in *Proceedings* a list of current engineering articles of interest. This was started in a modest way, and was evidently found useful by the membership, because a request soon came that it be printed on one side of the page only, in order that members might cut out items which specially interested them, and use them in their own indexes. This list, which has been published continuously in each

monthly number of the *Proceedings* from that date, is made up from an examination of about 115 periodicals. The classification is very simple, as the list is intended to be of current interest only, and to enable an engineer to glance over each month the publications relating to his particular line of work, and to select therefrom such articles as he may read either in some convenient library or by obtaining them from the publisher.

In order to show briefly the quantity of material written, edited, and published, the total number of pages issued in the Society publications for the twenty-five years from 1867 to 1892, was 17 747 (yearly average, 710), and for the twenty-five years from that time to date has been 96 800 (yearly average, 3 872), making the total pages 114 547. The cost of the printing, binding, and postage (nearly all the postage being chargeable to publications) for the latter period has been about $724 000 (yearly average, $28 960).

The actual handling, preparation for mailing, and mailing, of all these publications has been done by the Society force during that period.

In 1911 the writer presented a Report to the Board of Direction, and subsequently to the Business Meeting of the Annual Convention of that year, suggesting that there would be many advantages if a change were made in the method of getting out our publications. The report stated that he had investigated this possibility for some time and recommended that it be tried. Briefly, the idea was to continue the publication of *Proceedings* as heretofore, but to publish only one volume of *Transactions* per annum, such volume to contain as much matter as the four that were issued at that time. This was to be accomplished by the use of thin "India", or, as it is commonly called, "Bible", paper. Up to 1908 two volumes of *Transactions* had been issued yearly, but, beginning with 1909, four volumes were issued per annum. (In 1910 five volumes were issued.) These volumes contained between 550 and 600 pages each. The direct benefits were fully stated in this Report.*

The recommendation was approved and the first of these thin-paper volumes was issued in 1912.

It may be set down as axiomatic in Society work that no matter what may be done, it will not please the entire membership, and this case was no exception. So many criticisms were received, with in-

* *Proceedings*, Am. Soc. C. E., Vol. XXXVII, p. 319.

quiries as to why the Society had adopted the use of "tissue" paper in its publications, etc., etc., that in April, 1914, a circular was issued asking two questions:

(a) "Shall the use of thin paper be continued in the monthly *Proceedings?*

(b) "Shall the use of thin paper be continued in the one yearly volume of *Transactions,* or shall the same number of pages be issued in *Transactions* on thick paper, in four volumes per annum"?

The result of this was that, in a very large vote of about 3 000, 90% of those voting was in favor of the use of thin paper in the monthly *Proceedings,* and 95% was in favor of its use in *Transactions.*

As was foreseen, the points that appealed to the membership were the great saving to individuals in shelf room, in the cost of binding, and in economy in time by the use of one index instead of four.

Annual Conventions.

An Annual Convention has been held each year during the last twenty-five years, except in 1917, when the Convention which was to have been held in Minneapolis and St. Paul was abandoned on account of the war. Twenty-one separate localities have been visited. Two Conventions were held in Chicago, two in Niagara Falls, and two in San Francisco. All of them have been exceedingly enjoyable, have brought the members from various sections into closer contact, and have been of material benefit to individuals and to the Society.

It is perhaps worthy of notice that during this period three of these meetings have been held on the Pacific Coast, which up to 1896 was farther away from headquarters than the Society had ever held an official meeting, and that four were held on foreign soil, two in Canada, one in England, and one in Mexico.

It would extend this review too far even to touch upon the interesting events of these meetings, but perhaps it is permissible to call attention to the fact that the trip to London was made on the invitation of the Institution of Civil Engineers, that our meetings were held in the home of that Institution in London, and that the whole party had the honor and pleasure of being received by Queen Victoria at Windsor Castle. It might, perhaps, also be stated that the Mexican Convention was held by invitation of President Diaz. Members who

are interested will find quite full details of these trips in the *Proceedings.*

A special party was made up in March, 1911, to visit the Panama Canal. This was a more or less unofficial party. Two of the United Fruit Company's steamers were chartered for the occasion, one sailing from New York and the other from New Orleans, meeting at the Isthmus, and the party generally keeping together on the return. All the arrangements were made by the writer, who, unfortunately, was unable to go, due to the pressure of other duties, but he knows from what he heard from those who were fortunate enough to make it, that the trip was a specially enjoyable one.

SPECIAL COMMITTEES.

Reference should also be made to the splendid work of Special Committees appointed to investigate and report upon Engineering problems, twelve of which have made Final Reports during the period under consideration. The results of their work have been of inestimable value, but all that is possible, within the limits of this review, is to enumerate the subjects upon which such reports have been received.

Final Reports have been published on the following subjects:

Impurities in Public Water Supply; Standard Rail Sections—two Committees reported on this, one in 1893 and one in 1910—Uniform Methods for Testing Materials Used in Metallic Structures, and Requirements for These Materials to Further Improve the Grade of Such Structures; Standard Time; Regulating Practice of Engineering; Status of the Metric System in the United States; Uniform Tests of Cement; Conditions of Employment of, and Compensation of, Civil Engineers; Concrete and Reinforced Concrete; Principles and Methods for the Valuation of Railroad Property and Other Public Utilities; and Floods and Flood Prevention.

At the present time six Special Committees, all of which have presented one or more reports of progress, are investigating the following subjects:

Engineering Education; Steel Columns and Struts; Materials for Road Construction; Bearing Value of Soils for Foundations; Regulation of Water Rights; and Stresses in Railroad Track.

MEDALS AND PRIZES.

On October 1st, 1912, the Society established two additional prizes, as follows: The J. James R. Croes Medal, named in honor of the first recipient of the Norman Medal; and the James Laurie Prize, named in honor of the first President of the Society. The first consists of a medal of the value of $40, and may be awarded annually to such paper as may be judged worthy, and be next in order of merit to the paper to which the Norman Medal is awarded; the second consists of $40 in cash, with an engraved certificate signed by the President and by the Secretary of the Society. This prize also may be awarded annually, under the rules governing the award of the Thomas Fitch Rowland Prize, to such paper as may be judged worthy and be next in order of merit to the paper to which the Thomas Fitch Rowland Prize is awarded.

In a recent issue of *Engineering News-Record* the following editorial appears:

"AND THEY ARE FIGHTING IN FRANCE"

"The 'Subsidence of Muck and Peat Soils in Southern Louisiana and Florida' was the title of a paper presented two weeks ago at the meeting of the American Society of Civil Engineers. With the exception of three war addresses, equally peaceful topics have occupied the meetings since last April. The fall program, so far as announced, contains no papers bearing on the tremendous industrial and engineering problems which the winning of the war demands that we solve. This is an engineering war, yet the society seems not to recognize its opportunity."

It is unfortunate that such an improper, unfounded and sarcastic editorial insinuation should be made about an organization whose aims and objects are clearly unselfish, in a commercial publication on which the Profession in a large measure depends for its technical news.

The time for this attack upon the loyalty of this Society—just after it has become one of the Founder Societies—leaves an impression of malicious intent.

Of late all of us have heard much of the use of previously unheard of methods of warfare, and the writer feels sure that every right-minded member of our Allies of the Mining, Mechanical and Electrical Societies will unite with the members of this Society in condemnation of this misuse of editorial prerogative.

It is hoped that the following brief statement—written before the appearance of this insult to the Board of Direction and to the Membership of this Society—will be a sufficient answer.

WAR ACTIVITIES.

As soon as war was declared, the Society placed its facilities at the disposal of the Government, and, both as an individual organization and jointly with the other Founder Societies, has done all it has been asked or permitted to do.

The value of the Engineer has been recognized to a greater extent than ever before, and in the wonderful progress made in raising, training, transporting, and maintaining the new Army of the United States, as well as in the investigation and solution of new problems, he has been a most important factor.

A Joint Committee representing the National Societies, of which William Barclay Parsons, M. Am. Soc. C. E., was Chairman, was active in securing the legislation which provided for the Engineer Reserve Corps, and Committees made up of members of these Societies have been instrumental in recruiting Engineer Regiments in many parts of the country.

In 1915, in the absence of President Marx, the writer was requested by a Sub-committee of the Naval Consulting Board to co-operate with it and representatives of other National Societies, to formulate plans for industrial preparedness. He reported to the Board of Direction, on January 17th, 1916, that a plan had been developed by the Sub-committee, acting in conjunction with the five National Societies representing the Civil, Mining, Mechanical, Electrical, and Chemical Engineers, for securing complete statistics of the industrial strength of the country. Under this plan, in each State of the Union, one representative, recommended by each of these Societies, was appointed as an Associate Member of the Naval Consulting Board, and the five Engineers thus appointed in each State constituted a Board to secure the necessary information for the Government through the aid of the more than 30 000 members of these organizations. As is well known, this great work was carried to a successful conclusion.

In these and in many other ways the Society, and its Board of Direction, has been active in the present emergency.

Every member of the Society must read with pride our "Roll of Honor", the first issue of which,* incomplete as it undoubtedly is, contains the names of 575 Engineer officers who are now serving in the Army and Navy. Since that list went to press, 148 have been added to it, and it is still incomplete. This means that more than 8½% of the entire membership wears a uniform. The list, however, does not contain the names of hundreds of other members who are serving their country unobtrusively but still no less unselfishly and effectively, on Advisory Boards or simply as citizens. The writer knows of many cases where at great personal sacrifice such work has been and is now being done.

Only a few days ago a suggestion was made somewhat timidly over the telephone by J. W. DuB. Gould (one of our Members who is devoting his time to the service of the Government but who is one of those mentioned as not listed on our "Roll of Honor") that perhaps the Society might consider some arrangement by which the United States Food Administration could secure the use of the House we so recently vacated in order to carry on its work in New York City and State. The writer at once said that he believed that the Society would be glad to offer this House for the use of the Nation, for the purpose specified, free of charge.

It was not possible to get the Board together; indeed, in these busy times, a meeting of the Executive Committee is difficult to secure. By telephone, however, each available member of that Committee has given his unqualified and enthusiastic support to the proposition; the arrangement has been made, and the U. S. Food Administration Board will begin work at our old home on Friday of this week.

It is perhaps unnecessary to state that the head of this most important Board is a Member of this Society—Herbert C. Hoover.

Society Staff.

Any statement of the activities of the Society would be incomplete without special mention of the staff of the Secretary. It is not a large one. Before the transfer of the Library the total number (exclusive of Janitors and Office Boys) was 22; since that time it has been somewhat reduced. T. J. McMinn, M. Am. Soc. C. E., Assistant Secretary, and Miss Eleanor H. Frick, Chief Office Assistant, have served the Society for twenty years, and fourteen others for periods varying from

* *Proceedings*, Am. Soc. C. E., Vol. XLIII, p. 698 (November, 1917).

18 to 3 years, the average length of service of the entire force being more than 11 years. The Society owes much to the work of its employees, and the writer wishes to acknowledge publicly the faithful, industrious, efficient, and loyal service which has been rendered to the Society at all times, as well as to express his personal obligation to each of them.

Functions of a National Technical Society.

The writer believes that the primary functions of a National Technical Society might be stated about as follows:

1—To advance engineering knowledge and practice.

2—To maintain the dignity and standing of the organization, and to preserve the high character and professional qualifications of its membership.

3—To keep in touch with, and to take proper action on, all matters in which the relation of the Profession to the public is involved, and to render service to the Nation when occasion demands.

4—To do whatever is possible for its Members individually, and, in general, to return to them an equivalent for the dues paid.

The latter function necessarily takes the form of providing opportunity for professional discussion, both formal and informal, which, when, as is the case in this Society, more than 80% of the membership is non-resident, must be through publications.

The use of the Library should be brought as far as possible within the reach of all, and all matters brought to the attention of the management by correspondence should be handled promptly and efficiently. including the keeping of special records of members seeking professional engagements in order that they may be placed at the disposal of inquirers for technical men in any specialty.*

Perhaps the most difficult problem is to succeed in making each member feel that he is getting as much benefit as every other member. The men who framed the Constitution of the Society were wise enough to make a decided difference in the amount of dues to be paid by Resident and Non-Resident Members, but, although the Resident Member pays 66% more than the Non-Resident, the latter is still inclined to feel that those who live near Headquarters derive disproportionate benefits, in that they may attend all meetings, use the Reading Room, con-

* Though the Society has not advertised as an employment bureau, this plan has been in use for many years, and hundreds of members have been put in touch with professional opportunities.

sult the Library, and otherwise avail themselves of all local privileges.

It is not possible, of course, to arrange matters so that the Non-Resident can secure all these privileges, but, during the past twenty-five years, every effort has been made to do away with this feeling. How successful these efforts have been must be left to the individual judgment of each member, and it is hoped that what has been herein set down will aid in the formation of that judgment.

CO-OPERATION.

Why did this Society move its Headquarters? It occupied, as has been shown, a dignified, satisfactory, commodious House, in an excellent location, which was fully paid for; its standing as an organization left nothing to be desired; its membership was increasing rapidly in all parts of the country. Why, then, give up that which had been achieved by many years of unremitting effort?

It seems to the writer that the answer is that it was the right thing to do. What if, as an organization, some sacrifices were made? What if certain details of the movement did not appeal to certain individuals? Was it, or was it not, the thing to do, from the standpoint of the Engineering Profession? The best answer to these enquiries appears to be the vote of the membership, which was 2 500 in favor of, and only 390 against the change.

Since the inception of this co-operative movement the writer has been intimately associated with it, and in close contact with the men chosen by the Founder Societies to represent the other branches of our great Profession, and can testify that the most broad-minded, earnest, and sincere spirit of co-operation has been manifest.

In a report to the Board of Direction dated September 20th, 1915, the writer said:

"The value of unity of action in all matters which affect the Profession generally must be conceded.

"For many years the undersigned has been endeavoring to bring about such a condition; he has served on the John Fritz Medal Board of Award since its organization, and as its Executive Officer for 8 or 9 years; and is now its Chairman; has, with Mr. Ridgway, represented our Society on a joint committee for the consideration of a number of subjects * * *. He has actively represented the Society on the Committee of Management of the International Engineering Congress, and has been honored by the United Engineering Society by election to, and is now serving on, the Engineering Foundation Board.

"This experience has convinced him that there should be a permanent Board or Committee, composed of an equal number of representatives of the four National Societies, to which the duty of representing the 30 000 professional men now enrolled in their membership should be given. There are many ways in which such a representative body could help the status of the engineer, in his relations with clients, employers, and the public generally, which cannot, for obvious reasons, be taken up by any one of the Professional Societies individually, and it has been his thought that an organization now exists (the United Engineering Society) which, if the representatives of the Civil Engineer are added, and its powers somewhat expanded, would be ideal for the purpose. He now believes that this matter should be the subject of discussion between the Committees of this Society and of the United Engineering Society and that the result of their deliberation should be made part of the question to be submitted to all the organizations concerned."

Two years have elapsed since this was written, and without doubt the establishment of the "Engineering Council" was intended to provide for this long felt want. Although, up to the present time, the writer has seen no reason for changing the opinion expressed—that the United Engineering Society is the organization best fitted to act on these most vital matters—it is hoped and expected that the new body will prove its value.

The years covered by this review have been indeed busy ones, not without times of serious difficulty and trial, but the bright spots after all have predominated. Association with the leaders of thought along Engineering and Scientific lines is always broadening and helpful, and the writer looks back with pleasure only on the twenty-six years devoted to the service of the American Society of Civil Engineers, during twenty-three of which he has had the honor to be its Executive Officer and a member of its Board of Direction.

APPENDIX A

CLASSIFICATION OF THE LIBRARY OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS 1898–1916

BY

CHAS. WARREN HUNT

Secretary

A—RAILROADS
- Aa General
- Ab Location
- Ac Construction
- Ad Equipment
- Ae Operation
- Af Legal Documents
- Ag Reports, Company
- Ah Reports, State
- Ai Reports, Government
- Aj History

B—RAILROADS, STREET
- Ba General
- Bb Location
- Bc Construction
- Bd Equipment
- Be Operation
- Bf Legal Documents
- Bg Reports, Company
- Bh Reports, City
- Bi Reports, State
- Bj History

C—WATERWAYS
- Ca General
- Cb Rivers
- Cc Harbors
- Cd Lakes
- Ce Oceans
- Cf Canals
  - Cfa History
  - Cfb Location
  - Cfc Construction
  - Cfd Equipment
  - Cfe Operation
  - Cff Legal Documents
  - Cfg Reports, Company
  - Cfh Reports, State
  - Cfi Reports, Government

D—WATER SUPPLY
- Da General
- Db Water
- Dc Works
- Dd Power
- De Irrigation

E—SANITATION
- Ea General
- Eb Drainage
- Ec Sewerage
- Ed House Drainage
- Ee Sewage Disposal
- Ef Garbage Disposal
- Eg Health and Disease
- Eh Ventilation and Heating

F—BRIDGES
- Fa General
- Fb Arch
- Fc Cantilever
- Fd Draw
- Fe Girder
- Ff Lift
- Fg Suspension
- Fh Truss
- Fi Viaducts

G—MECHANICAL
- Ga General
- Gb Hydraulic Machinery
- Gc Steam Engines
- Gd Boilers
- Ge Compressed Air

H—ELECTRIC
- Ha General
- Hb Light
- Hc Power
- Hd Telegraph
- He Telephone
- Hf Various Uses

I—GAS
- Ia General
- Ib Coal
- Ic Natural
- Id Water

J—ARCHITECTURE AND BUILDING
- Ja General
- Jb Buildings
- Jc Materials
- Jd Laws
- Je Fire Prevention

K—MARINE
- Ka General
- Kb Yards
- Kc Ordnance
- Kd Naval Ships
- Ke Merchant Ships
- Kf Steam Boats

L—MILITARY
- La General
- Lb Tactics
- Lc Fortifications
- Ld Ordnance

M—MINING
- Ma General
- Mb Coal
- Mc Copper
- Md Gold and Silver
- Me Iron

N—ROADS AND PAVEMENTS
- Na General
- Nb Earth
- Nc Broken Stone
- Nd Plank
- Ne Monolithic
- Nf Brick
- Nh Stone Block
- Ni Wooden Block

O—MUNICIPAL REPORTS

P—LANDSCAPE ARCHITECTURE

Q—GEOGRAPHY
- Qa General
- Qb Physical
- Qc Statistics
- Qd Resources
- Qe Surveys
- Qf Maps and Atlases

R—SURVEYING AND DRAWING

S—SOCIETY PUBLICATIONS
- Sa—North America:
  - Sa1 Canada
  - Sa2 Mexico
  - Sa3 United States
- Sb—South America:
  - Sb1 Argentine Republic
  - Sb4 Chile
  - Sb11 Venezuela
- Sc—Central America
- Sd—Europe:
  - Sd1 Austria
  - Sd2 Belgium
  - Sd3 Denmark
  - Sd4 France
  - Sd5 Germany
  - Sd6 Great Britain
  - Sd8 Italy
  - Sd9 Netherlands
  - Sd10 Norway
  - Sd11 Portugal
  - Sd13 Russia
  - Sd14 Spain
  - Sd15 Sweden
  - Sd16 Switzerland
- Se—Asia
- Sf—Africa
- Sg—Australia

T—PERIODICALS
- Ta—North America:
  - Ta2 Mexico
  - Ta3 United States
  - Ta4 West Indies
- Tb—South America:
  - Tb1 Argentine Republic
  - Tb3 Brazil
  - Tb5 Colombia
  - Tb6 Ecuador
  - Tb9 Peru
  - Tb11 Venezuela
- Td—Europe:
  - Td1 Austria
  - Td2 Belgium
  - Td3 Denmark
  - Td4 France
  - Td5 Germany
  - Td6 Great Britain
  - Td7 Hungary
  - Td8 Italy
  - Td9 Netherlands
  - Td10 Norway
  - Td13 Russia
  - Td14 Spain
  - Td15 Sweden
- Te—Asia
- Tg—Australia

U—DICTIONARIES AND ENCYCLOPEDIAS

V—ENGINEERING HANDBOOKS

Y—GENERAL SCIENCE
- Ya General
- Yb Agriculture and Forestry
- Yc Astronomy
- Yc5 Biology
- Yc9 Botany
- Yd Chemistry
- Ye Education
- Yf Exhibitions
- Yh Geology
- Yh9 Mathematics
- Yi Metallurgy
- Yj Meteorology
- Yk Patents
- Yl Physics
- Ym Weights and Measures
- Yn Zoology

Z—MISCELLANEOUS
- Za General
- Za5 Archaeology
- Zb Biography
- Zc Charities and Corrections
- Zd Commerce
- Ze Fine Arts
- Zf Fisheries
- Zg History
- Zh Law
- Zi Manufactures
- Zj Political Economy
- Zk Religion

APPENDIX B

PROPOSED CLASSIFICATION

FOR AN

ENGINEERING LIBRARY

COMPILED BY ELEANOR H. FRICK AND ESTHER RAYMOND

OF THE LIBRARY STAFF OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

UNDER DIRECTION OF THE SECRETARY

1916

Accompanying "The Activities of the American Society of Civil Engineers During the Past Twenty-five Years", by Chas. Warren Hunt, M. Am. Soc. C. E.

EXPLANATORY

Civil Engineering is the only class which has been expanded in detail. Certain subjects have, of necessity, been classed arbitrarily, the principal thought being utility. For instance, "Water Wheels" are placed under "Water Power" rather than under "Hydraulic Machinery"—"Locomotives" under "Railroads" rather than under "Steam Engines".

ACKNOWLEDGMENT

To acknowledge every source used in compiling this classification is not possible, because hundreds of books and indexes have been examined; mention should, however, be made of unpublished material of the Joint Committee on Classification of Technical Literature; publications of the Library of Congress, University of Illinois Extension of Dewey, and the Dewey Decimal Classifications. To John M. Goodell, and Henry S. Jacoby, Associates, Am. Soc. C. E., T. J. McMinn, and A. H. Van Cleve, Members, Am. Soc. C. E., Mr. H. E. Haferkorn, Librarian, U. S. Engineer School, Washington Barracks, and members of the Special Committee on Materials for Road Construction, of the American Society of Civil Engineers, special acknowledgment is made.

CLASSES

000 GENERAL
100 CIVIL ENGINEERING
200 MECHANICAL ENGINEERING
300 ELECTRICAL ENGINEERING
400 MINING ENGINEERING
500 METALLURGY
600 GAS ENGINEERING
700 CHEMICAL TECHNOLOGY. MANUFACTURES
800 MILITARY AND NAVAL SCIENCE
900 OTHER SUBJECTS

DIVISIONS

000 GENERAL
010 Engineering Bibliographies
020 Engineering Encyclopedias
030 Engineering Dictionaries
040 Engineering Directories
050 Engineering Societies
060 Engineering Periodicals
070 Patents
080 Engineering in General
090 Materials of Engineering

100 CIVIL ENGINEERING
110 Structural Engineering. Bridges. Buildings
120 Surveying
130 Railroads
140 Street Railroads
150 Highways
160 Hydrology. Hydraulics. Dams
170 Waterways
180 Water Power. Water-Works. Irrigation. Drainage
190 Sanitation

200 MECHANICAL ENGINEERING
210 Power Transmission. Millwork
220 Heat Engineering
230 Automobiles
240 Aeronautics
250 Hydraulic Machinery
260 Machinery for Special Purposes
270 Machine Shops
280 Miscellaneous Types of Power

300 ELECTRICAL ENGINEERING
310 Electric Measurement
320 Dynamo-Electric Machinery
330 Control
340 Transmission
350 Telephone
360 Telegraph
370 Lighting
380 Chemical Electricity. Batteries.
390 Other Uses

400 MINING ENGINEERING
410 Prospecting. Mine Surveying
420 Excavation and Working
430 Drainage and Sanitation
440 Transportation
450 Ventilation
460 Lighting. Signaling
470 Electricity in Mining
480 Accidents. Safety Measure
490 Mining Special Kinds of Ore

500 METALLURGY
510 Iron and Steel
520 Gold and Silver
530 Copper
540 Lead
550 Tin
560 Zinc
580 Other Metals
590 Assaying

600 GAS ENGINEERING
610 Natural Gas
620 Materials
630 Manufacture and Works
640 Storage
650 Distribution
660 Utilization
670 By Products
680 Management

700 CHEMICAL TECHNOLOGY. MANUFACTURES
710 Chemicals. Dyes. Paints
720 Ceramics
730 Metal Manufactures. Machinery
740 Lumbering. Wood Manufactures
750 Paper Making
760 Textiles
770 Leather Manufacture. Tanning
780 Foods and Beverages
790 Miscellaneous Industries

800 MILITARY AND NAVAL SCIENCE
810 Military Science. General
820 Fortifications
830 Ordnance
840 Naval Architecture. Shipbuilding Yards
860 Navigation. Shipping
870 Naval Science. War Vessels
880 Naval Strategy and Tactics
890 Naval Organization

900 OTHER SUBJECTS
910 Philosophy.
920 Religion
930 Sociology
940 Philology
950 Natural Science
960 Useful Arts (Other than Engineering and Manufactures)
970 Fine Arts
980 Literature
990 History

SUB-DIVISIONS

To Be Used With Any Class or Sub-Class

The following nine divisions have been used as the first general sub-divisions under each main class. They may also be used with sub-divisions of any class. For instance, the sub-division *Costs and Estimates* (.04) may be applied to the general subject of Electrical Engineering (300.04), and may also be used under *Dynamo-Electric Machinery* (320), a sub-division of Electrical Engineering (320.04), and also under *Dynamotors* (322.3), which is a sub-division of *Dynamo-Electric Machinery* (322.304)—*Dynamo-Electric Machinery* being 320, sub-division *Direct-Current Machinery* being 322, and sub-division *Dynamotors* being 322.3.

.01 History
.02 Laws and Legislation
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories

000 GENERAL
010 Engineering Bibliographies
020 Engineering Encyclopedias
030 Engineering Dictionaries
040 Engineering Directories
050 Engineering Societies
060 Engineering Periodicals
070 Patents
080 Engineering in General
081 General Works
085 Ethics
086 Valuation of Utilities (For Valuation of a special utility see special subject)
087 Industrial Management
.1 Organization
.2 Efficiency Engineering
.21 Scientific Management. Motion Study
088 Construction Work. Contracting
.1 Contracts and Specifications. General Works (For Special Contracts and Specifications, *See* .05, under that subject)
.2 Organization
.3 Methods
.31 Timekeeping,etc.
.4 Inspection
.5 Contractors' Plant
089 Excavation. Earthwork (*See also* 420, Excavation and Working under Mining Engineering)
.1 Earth Excavation
.2 Rock Excavation
.3 Excavating Machinery
.31 Steam Shovels
.32 Ditching and Trenching Machinery
090 Materials of Engineering (*See also* 111, Mechanics of Materials)
091 Engineering and Testing Laboratories
.1 Laboratory Manuals
.2 Testing Machines and Appliances
.3 Methods of Testing

091	**Engineering and Testing Laboratories** (*Continued*)
.31	Selection of Test Pieces. Influence of Temperatures, etc.
.32	Weathering
.33	Elastic Limit Tests
.34	Tension, Compression, Torsion, Flexure, Shearing
.341	Tensile Tests
.342	Compression Tests
.343	Torsion Tests
.344	Flexure Tests
.345	Shearing Tests
.346	Repeated Stress Tests
.35	Impact. Repeated Shock Tests
.36	Hardness Tests
.37	Special Tests (Varying for different materials)
.38	Tests on Special Shapes and Forms
.39	Other Tests
092	**Timber. Strength and Testing**
.1	Influence of Temperature
.2	Weathering. Decay and Preservation
.3	Elastic Limit Tests
.4	Tension, Compression, Torsion, Flexure, Shearing
.5	Impact. Repeated Shock Tests
.6	Hardness Tests
.7	Special Tests for Timber
.8	Special Shapes
.81	Posts
.82	Columns
.83	Shafts
.84	Cylinders. Pipe, etc.
.9	Descriptions of Various Kinds of Timber (Arranged Alphabetically)
093	**Masonry Materials**
.1	Stone
.11	Influence of Temperature
.12	Weathering
.14	Tension, Compression, Shearing, Crushing
.15	Impact
.16	Hardness Tests
.17	Special Tests for Stone, etc.
.18	Special Shapes and Forms
.19	Descriptions of Kinds of Stone (Arranged Alphabetically)
.2	Brick
.21	Influence of Temperature
.22	Weathering
.24	Crushing Tests
.27	Special Tests
.271	Rattler Tests
.272	Absorption Tests
.3	Tile
.4	Terra Cotta
.5	Lime. Mortar
.6	Cement
.61	Influence of Temperature. Selection of Test Pieces
.62	Weathering
.63	Tension, Compression, etc.
.65	Impact Tests
.66	Soundness. Constancy and Time of Setting
.67	Special Tests
.671	Fineness of Grinding
.672	Accelerated Tests
094	**Concrete. Strength and Testing**
.1	Influence of Temperature and Mixing. Selection of Pieces
.11	Concrete Aggregates
.111	Sand
.112	Gravel
.113	Slag
.114	Water
.12	Forms. Removal of Forms
.13	Effects of Freezing on Concrete
.2	Weathering. Concrete Finishing
.21	Action of Salt Water
.22	Action of Gases and Other Chemicals
.23	Water-proofing
.24	Concrete Finishes. Stucco
.3	Elastic Limit Tests
.4	Tension, Compression, Torsion, Flexure, Shearing
.5	Impact
.6	Hardness

094	Concrete. Strength and Testing (*Continued*)
.7	Special Tests for Concrete
.8	Special Shapes
.81	Bars. Slabs
.82	Columns. Posts. Footings
095	Reinforced Concrete. Strength and Testing
.1	Influence of Temperature. Expansion and Contraction
.2	Weathering. Corrosion of Reinforcement
.3	Elastic Limit Tests
.4	Tension, Compression, Torsion, Flexure, Shearing
.5	Impact. Repeated Shock Tests
.6	Hardness
.7	Special Tests
.71	Bond of Concrete and Metal
.8	Special Shapes
.81	Bars. Beams. Slabs
.82	Columns. Posts. Struts. Footings
.83	Shafts
.84	Cylinders. Pipes, etc.
.9	Types of Reinforcement
096	Other Non-Metallic Materials (Glass, Asbestos, etc.)
097	Iron and Steel
.1	Effect of Temperature
.2	Weathering. Corrosion and Protection
.3	Elastic Limit Tests
.4	Tension, Compression, Torsion, Flexure, Shearing
.5	Impact
.6	Hardness
.7	Special Tests
.8	Special Shapes
.81	Bars. Beams
.811	Continuous Beams
.82	Columns
.83	Shafts
.84	Cylinders. Rollers. Spheres. Tubes. Pipes
.85	Rivets. Pins. Riveted Joints
.86	Nails. Screws. Bolts
.87	Angles. Shapes. Plates
.89	Other Shapes
.891	Springs
.892	Wire
.893	Chains. Chain Links
.9	Special Materials
.91	Cast Iron
.92	Wrought Iron
.93	Steel
098	Metals (Other than Iron and Steel)
.1	Copper
.2	Zinc
.9	Alloys
099	Other Materials
100	CIVIL ENGINEERING
	General
.01	History
.02	Laws and Legislation
.03	Statistics
.04	Costs and Estimates
.05	Contracts and Specifications
.06	Drawings
.07	Congresses
.08	Exhibitions
.09	Tests. Laboratories (*See also* 090, Materials of Engineering)
110	Structural Engineering. Bridges. Buildings
111	Mechanics of Materials. Strength of Materials. General Theory
.1	Beams
.11	Continuous Beams
.2	Columns. Posts. Struts
.3	Shafts
.4	Cylinders. Spheres
.5	Rollers. Plates
.6	Riveted Joints. Pin Proportioning
112	Theory of Design. Stresses and Strains
.1	Analytic Methods. Graphic Statics
.2	Loads and Stresses (General Material here. For Stresses in a Special Structure, *See* 112.9)
.21	Dead Loads
.22	Live Loads

112 Theory of Design. Stresses and Strains ***(Continued)***
.23 **Impact Loads**
.24 **Wind Pressure**
.25 **Snow Loads**
.3 **Iron and Steel Construction (General. For an Iron Truss Bridge, (*See* 112.92)**
.4 **Masonry Construction (General)**
.41 **Stone Construction (General)**
.42 **Brick Construction (General)**
.5 **Concrete Construction (General)**
.6 **Reinforced Concrete Construction (General)**
.7 **Timber Construction (General)**
.8 **Esthetics of Design**
.9 **Design of Special Types**
.91 **Arches. Arch Bridges**
.911 **Metallic**
.912 **Masonry**
.913 **Concrete**
.914 **Reinforced Concrete**
.92 **Trusses. Truss Bridges**
.921 **Simple**
.922 **Continuous**
.93 **Girders. Girder Bridges**
.94 **Movable Bridges**
.941 **Swing**
.942 **Lift. Bascule**
Turntables (*See* 133.83)
.95 **Cantilever Bridges**
.96 **Suspension Bridges**
.97 **Viaducts**
.98 **Other Forms of Bridges**
.981 **Ferry Bridges. Transporter Bridges**
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.0012 **Supporting Power of Soils**
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.213 **Concrete**
.214 **Reinforced Concrete**
.22 **Timber Grillage**
.23 **Steel Beam Grillage**
.24 **Eccentric Footing**
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.26 **Inverted Arch**
.3 **Piles and Pile-Driving**
.31 **Timber Piles**
.311 **Mechanical Protection (Cement Mortar, Galvanized Iron, Pipe, etc.)**
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.32 **Concrete Piles**
.321 **Pre-moulded**
.322 **Cast in Place**
.33 **Metal Piles**
.331 **Disk Piles**
.332 **Screw Piles**
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.34 **Sheet-Piles**
.35 **Pile-Drivers**
.36 **Driving by Water-Jet**
.37 **Bearing Power of Piles**
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.5 **Rails**
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.7 **Removal of Wrecks, Snow, and Weeds**
.71 **Removal of Wrecks**
.72 **Removal of Snow**
.73 **Removal of Weeds**
.8 **Track Accessories**
.81 **Track Tanks. Fuel and Water Stations**
.82 **Ash-pits, etc.**
.83 **Turntables. Transfer Tables**
.84 **Fences, Cattle-guards, Snow Guards**
.85 **Snow Sheds**
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134 **Buildings**
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.4 **Shops**
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135 **Rolling Stock**
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.111 **Wheels**
.112 **Axles**
.113 **Bearings and Lubrication**
.114 **Springs**
.115 **Couplings**
.116 **Brakes**
.117 **Furnishings**
.118 **Sanitation, Heating, Lighting, Ventilation**
.12 **Passenger Cars**
.13 **Baggage and Mail Cars**
.14 **Freight Cars**
.15 **Work Cars**
Motor Cars (*See* 136.6)
.2 **Locomotives (Put here Manuals for Locomotive Engineers and Firemen. *See also* Electric Locomotives, 136.5)**

135 **Rolling Stock** (*Continued*)
.21 **Design and Construction**
.22 **Details (Frames, Boilers, etc. Divide if necessary)**
.23 **Types**
.24 **Locomotive Tenders**
.25 **Locomotive Shops and Works**
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.27 **Locomotive Performance**
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.23 **Other Systems**
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.5 **Electric Locomotives**
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.71 **Motors, Controllers, etc.**
.8 **Locomotive and Car Wiring**
137 **Operation**
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.11 **Hand Signals**
.12 **Telegraph and Telephone Systems**
.13 **Block Systems**
.14 **Interlocking Systems**
.2 **Train Movement**
.21 **Train Resistance**
.22 **Train Speed**
.23 **Train Load**
.24 **Train Running**
.241 **Travelers' Guides. Time-Tables**
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.4 **Traffic**
.41 **Station Management**
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.44 **Express**
.45 **Mail Service**
.46 **Freight**
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.51 **Passenger Tariffs**
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.61 **Capitalization**
.62 **Valuation**
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.64 **Receivership and Reorganization**
.65 **Clearing House**
.7 **Organization of Staff and Force**
.71 **Service Rules and Regulations**
.72 **Wages**
.73 **Strikes**
.8 **Federal and State Relations**
138 **Reports**
.1 **Company Reports**
.2 **State Commission Reports (Enter here only serials or reports so general in nature that they cannot be classed elsewhere)**
.3 **Government Reports (Above note applies also to this class)**
139 **Miscellaneous Kinds of Railways**
.1 **Mountain**
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.2 **Monorail**
.3 **Aerial Tramways**
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140 Street Railways. Elevated Railways. Subways (*Continued*)
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144 Structures and Buildings
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.3 Shops
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145 Rolling Stock
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.5 Accessories and Parts. Street Railway Motors, etc.
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.1 Electric Power Requirements
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150 Highways (General Works and Treatises classed here)
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.02 Legislation
.021 Municipal
.022 County
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151	**Administration. Reports**
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.8	**Bridges and Fords (Location only. For Design and Construction of Bridges, *See* 110)**
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.33	**Tar Concrete**
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.37	**Stone Blocks**
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.4	**Curbs**
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.42	**Cement-Concrete**
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.8	**Trackways**
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	Tree Culture (*See* 963, Agriculture and Forestry)
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155	**Grading, Drainage and Foundations**
.1	**Grading**
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.97 For Special Purposes
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.972 Race Tracks
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.13 Scrapers
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161 Hydrology. Water Resources
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.2 Rainfall and Run-off
.3 Stream Measurements
.4 Springs. Wells
.5 Underground Water
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.14 Floats (For Velocities and Directions)
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.8 **Federal and State Relations (General only. Special reports go with State and Government Reports)**
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.11 **Storage and Regulation Reservoirs (Put here general discussion of the subject. For theory of design and construction, *See* 160)**
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.13 **Bank and Bed Protection**
.131 **Training Walls**
.132 **Spur-Dikes, Groins, Hurdles, etc.**
.133 **Ground Sills**
.134 **Levees (Including Embankments and Dikes)**
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.14 **River Outlets (Embouchures)**
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181	**Water Power. Hydro-electric Plants**
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.211	Racks
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.22	In Reservoirs (For Construction, *See* 164, Dams and Reservoirs, under Hydrology. Hydraulics. Dams)
.23	Through Intakes
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183	**Analysis. Quality** (*See also* 161.1, Physical Properties of Water, under Hydrology. Hydraulics. Dams)
184	**Purification of Water**
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.47 **Other Kinds of Mains**
.48 **Coatings. Water-proofing**
.49 **Accessories. Gates, Valves, etc.**
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.16 **Preliminary Investigation. Surveys. Population**
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.311	**Circular**
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.347	**Concrete**
.3471	**Concrete Block**
.348	**Reinforced Concrete**
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.41	**Openings in Streets**
.42	**Other Openings**
.5	**Sewer Appurtenances**
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.52	**Traps**
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.54	**Catch-Basins. Inlets**
.55	**Flush-Tanks. Flushing Fixtures**
.551	**Automatic Siphons**
.56	**Outfalls. Outlets**
.561	**Sea Outfalls**
.57	**Special Valves. Tide Gates**
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.572	**Overflows. Interceptors**
.573	**Relief Chambers**
.58	**House Connections**
.59	**Other Appurtenances**
.591	**Covers, Frames, Gratings, Ladders, Steps**
.592	**Sewage Reservoirs. Storage Tanks**
.6	**Construction**
.61	**Excavation**
	Excavating Machinery (*See* **89.32, Ditching and Trenching Machinery, under Excavation)**
.62	**Timbering**
.63	**Pumping and Draining Sewer Trenches**
.64	**Foundations**
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.11	**Character and Composition**
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.19	**Bacteria in Their Relation to Sewage Disposal**
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.311	**Coarse Screens**
.312	**Fixed**
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.323	**Horizontal-Flow Tanks**
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.353	**Drying by Centrifuging Machines**
.354	**Draining on Beds**
.355	**Pressing**
.356	**Burying**
.357	**Burning**
.359	**Other Methods**
.36	**Sludge Utilization**
.361	**Fertilizer**
.362	**Grease**
.363	**Fuel**
.37	**Irrigation**
.371	**Sub-surface Irrigation**
.372	**Broad Irrigation**
.38	**Sterilization and Disinfection**
.381	**Electrolytic Processes**
.4	**Cess-pools**
.5	**Pail Systems**
.6	**Disposal for Isolated Plants**
.61	**Camp Sanitation**
.9	**Reports on Sewage Disposal of Special Places**

194 **Refuse Disposal**
.1 Garbage
.2 Ashes. Rubbish. Street Sweepings
.3 Animals
.4 Slaughter-House and Market Refuse
.5 Factory and Trade Wastes
.6 Incineration Plants
.7 Reduction Plants
197 **Air Pollution**
.1 Dust
.2 Gases
.3 Odors
Smoke (*See* 222.13)
198 **Industrial and Factory Sanitation**
Plumbing of Factories (*See* 192.18)
199 **Public Health. Vital Statistics**
.1 Food Regulations. Ice. Drugs
.11 Milk and Milk Products
.12 Animal and Plant Foods
.13 Ice
.14 Drugs
Housing Conditions (*See* 117.42)
.2 Fly and Mosquito Suppression
.3 Contagious and Infectious Diseases
.31 Epidemics
.32 Quarantine
.33 Disinfection
Hospitals (*See* 961 and 117.6)
.4 Disposal of the Dead
.41 Undertaking and Embalming
.42 Cremation
.8 Registration. Vital Statistics
.9 Boards of Health. Reports
.91 City
.92 State
.93 Government

200 **MECHANICAL ENGINEERING**
General
.01 History
.02 Laws and Legislation
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories
201 **Mechanics of Machines. Strength of Materials**
202 **Machine Drawing**
203 **Machine Parts**
.1 Fastenings. Bolts. Rivets. Screws
.2 Axles
.3 Cranks. Cams
.4 Springs
.5 Bearings
.6 Couplings
.7 Shafting
.9 Other Parts
204 **Lubrication. Friction**
205 **Balancing of Engines**
209 **Reports**
210 **Power Transmission. Mill Work**
211 **Links and Link Motion**
212 **Gearing**
213 **Belts and Pulleys**
214 **Chain Transmission**
220 **Heat Engineering**
221 **Thermodynamics. Theory of Heat Engines**
222 **Heat Generation**
.1 Combustion
.11 Chemistry
.12 Temperatures. Pyrometry
.13 Smoke. Smoke Prevention
.2 Fuel. Production, Analysis, and Use

222 **Heat Generation** (*Continued*)
.21 Coal
Coke (*See* 671, under Gas Engineering)
.22 Wood
.23 Charcoal
.24 Pulverized Fuel. Briquettes
.25 Liquid Fuel
.26 Gas Fuel
.29 Other Fuels
.3 Furnaces
.31 Draft
.32 Grates
.33 Stokers
.34 Ash-Burners
.35 Ash Disposal
.4 Liquid Fuel Apparatus
223 **Steam Engineering**
.1 Steam (Properties, etc.)
.2 Steam Boilers. Design. Construction.
223.21 Water Supply. Corrosion (For Water Softening. *See* 184.5, under Purification of Water)
.211 Feed-Water Heaters
.22 Details and Parts
.221 Rivets. Stays
.222 Tubes
.23 Boiler Accessories
.231 Superheaters
.24 Types (For Locomotive Boilers, *See* 135.22, under Rolling Stock)
.241 Fire-Tube Boilers
.242 Water-Tube Boilers
.243 Stationary Boilers
.244 Marine Boilers
.25 Operation and Economy
.26 Explosions. Accidents
.261 Boiler Inspection
.3 Steam Engines
.31 Details and Parts
.311 Valves and Valve Gears
.312 Cylinders
.313 Governors
.314 Fly-Wheels
.315 Indicators
.32 Steam Turbines
.33 Types (Not Classed Elsewhere)
.331 Steam Turbines
.332 Traction Engines
.333 Stationary Engines
.334 Portable Engines
.335 Rotary Engines
.336 Marine Engines
.4 Steam Power and Boiler Plants
.41 Piping
.42 Condensers
.43 Cooling Towers and Ponds
224 **Gas and Oil Engines. Internal Combustion Engines**
.1 Details and Parts
.11 Ignition
.12 Carburetion
.13 Governors
.14 Valves and Valve Gears
.2 Types
.21 Gas Engines
.22 Oil Engines
.23 Gasoline Engines
.24 Alcohol Engines
.3 Gas Power Plants
225 **Compressed Air**
226 **Refrigeration**
227 **Heating and Ventilation**
230 **Automobiles**
240 **Aeronautics**
250 **Hydraulic Machinery.** (For general theory of Hydraulics, *See* 160, Hydrology. Hydraulics. Dams. For Water Wheels and Turbines, *See* 181.26, under Water Power)
251 **Pumps and Pumping Engines** (*See also* 432, Mine Pumps)
252 **Hydraulic Presses**

253 Hydraulic Rams
Hydraulic Lifts. Elevators. Cranes (*See* 260)
260 Machinery for Special Purposes
261 Hoisting and Conveying Machinery
.1 Cranes and Derricks
.2 Cableways
.3 Coal Handling
.4 Ore Handling
262 Elevators
263 Agricultural Machinery
264 Drying Machinery
270 Machine Shops and Machine Shop Practice
271 Machine Shop Management
272 Machine Tools
.1 Planing Machines
.2 Grinding and Filing
.3 Cutting and Sawing
.4 Turning and Milling. Lathes
.5 Drills
.6 Punching and Shearing
.7 Bending. Straightening. Shaping
280 Miscellaneous. Types of Power
281 Animal Power
282 Solar Engines
283 Wind Power
.1 Windmills

300 ELECTRICAL ENGINEERING
General
.01 History
.02 Laws and Legislation
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories
306 Induction Coil
308 Economics
.1 Rates
.2 Valuation
.3 Contracting
309 Electric Utilities. Reports of Companies
310 Electric Measurement
311 Units and Standards
312 Meters
.1 Ammeter
.2 Voltmeter
.3 Ohmmeter
.4 Watt-meter
.5 Coulometer
.6 Potentiometer
.7 Galvanometer
.8 Wheatstone Bridge. Slide-Wire Bridge
.9 Other Meters
313 Current Measurement
314 Power Measurement
315 Resistance Measurement
316 Potential Measurement
317 Capacity Measurement
318 Inductance Measurement
319 Other Measurements
320 Dynamo-Electric Machinery. Transformers. Central Stations
.1 Armature and Field Windings
321 Electric Plants. Central Stations. (For Hydro-Electric Plants, *See* 181, under Water Power)
.1 Steam Driven
.2 Gas Driven
.3 Oil Driven
.4 Operation and Management
.41 Maintenance
.42 Rates
.43 Finance
.44 Valuation
.45 Accounting

322 Direct Currents and Machines
.1 Generators
.2 Motors
.3 Dynamotors
.4 Direct-Current Boosters
323 Alternating Currents and Machines
.1 Synchronous Machines
.11 Generators
.12 Motors
.13 Converters (Rotary Converters)
.2 Asynchronous Machines. Induction Machines
.21 Induction Generators
.211 Single-Phase
.212 Three-Phase
.22 Induction Motors
.221 Single-Phase
.222 Polyphase
.3 Motor-Converters
.4 Alternating-Current Commutator Motors
.41 Series Motors
.42 Repulsion Motors
324 Double-Current Generators
325 Motor Generators
326 Rectifying Apparatus
327 Stationary Induction Apparatus
.1 Transformers
.2 Auto-Transformers. Compensators
.3 Potential Regulators
.4 Reactors or Reactance Coils
328 Electrostatic Apparatus
.1 Condensers
330 Control Devices. Switches and Switch-boards
331 Switches
332 Switch-boards. Panels
333 Controllers. Rheostats
338 Wireless Control
339 Protective Devices
.1 Fuses
.2 Circuit Breakers
.3 Reactances
.4 Lightning Protection
340 Transmission. Distribution
341 Systems. Line Phenomena
.1 Direct Current
.2 Alternating Current
342 Transmission Materials
.1 Wire
.2 Cables
.3 Insulators
343 Overhead Lines
.1 Design of Overhead Lines. Catenary. Stresses. Wind Pressure
.2 Poles
.3 Towers
.4 Accessories. Cross-Arms. Pins
344 Underground Lines
345 Electric Wiring
350 Telephone
351 Line Transmission
352 Systems
353 Instruments
.1 Transmitter
.2 Receiver
354 Switch-boards and Other Station Equipment
357 Wireless Telephony
358 Operation. Rates
359 Telephone Companies. Reports
360 Telegraph
361 Line Transmission
362 Instruments
363 Systems. Codes
.1 Morse
.2 Hughes
.3 Simplex
.4 Multiplex
364 Printing Telegraphy
365 Picture Telegraphy

366 Submarine Cable Telegraphy
367 Wireless Telegraphy
368 Operation. Rates
369 Companies. Reports
370 Electric Lighting
371 Illumination
.1 Interior
.2 Exterior. Streets, Parks, etc.
372 Electric Lamps
.1 Arc Lamps
.11 Carbon
.12 Mineralized
.13 Vapor
2 Incandescent Filament Lamps
.21 Carbon and Carbon Metallized
.22 Metal
.23 Other
373 Electric Light Plants for Country Houses, etc.
374 Special Uses of Electric Light
.1 Searchlights
.2 Signs
380 Chemical Electricity. Batteries
381 Primary Cells
382 Storage Batteries. Accumulators
383 Lead Accumulators
390 Other Uses of Electricity
391 Industrial
.1 Electric Drive
Elevators, Cranes (*See* 260, under Mechanical Engineering)
392 Domestic
393 Therapeutic
394 Agricultural
Mining (*See* 470, under Mining Engineering)

400 MINING ENGINEERING
General
.01 History
.02 Laws and Legislation
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories
408 Economics. Mine Accounting
409 Organization and Administration
.1 Mine Labor
.2 Reports of Mine Departments and Inspectors
410 Prospecting. Mine Surveying
411 Prospecting
.1 Divining Rod
412 Mine Surveying
413 Mine Models
414 Sampling
415 Valuation
416 Prospectuses
420 Excavation and Working
421 Open Working
.1 Placer Mining
.11 Working with Pan
.12 Sluicing
.13 Hydraulic Mining
.14 Dredging
422 Drilling. Blasting
.1 Explosives
423 Quarrying
424 Boring
425 Shaft Sinking
.1 Shaft Lining
426 Tunneling and Drifting
427 Stoping
428 Timbering and Mine Supports. Masonry Lining
430 Drainage and Sanitation
431 Mine Waters

432 **Mine Pumps**
433 **Mine Dams**
434 **Sanitation**
440 **Transportation. Haulage and Hoisting**
441 **Underground Haulage**
442 **Mine Locomotives**
443 **Hoisting**
.1 **Hoisting Engines**
.2 **Cages. Skips. Buckets**
444 **Surface Transportation**
445 **Storage** (*See also* **117.52, Coal Storage Plants, under Building Construction**)
450 **Ventilation**
451 **Mine Gases. Fire-damp.**
460 **Lighting. Signaling**
461 **Safety Lamps**
462 **Signals and Signaling**
470 **Electricity in Mining**
480 **Accidents. Safety Measures and First Aid**
481 **Explosives**
.1 Dust Explosions
482 **Mine Fires**
483 **Safety Regulations**
484 **Mine Rescue Work. First Aid**
490 **Mining of Special Kinds of Ore**
491 **Coal Mining**
492 **Copper Mining**
493 **Gold and Silver Mining**
494 **Iron Mining**
495 **Lead Mining**
496 **Petroleum Mining**
497 **Tin Mining**
498 **Zinc Mining**
499 **Miscellaneous** (**Arranged Alphabetically by Metal,** *e. g.,* **Antimony, Cadmium, Cobalt, etc.**)

500 **METALLURGY**
General
.01 **History**
.02 **Laws and Legislation**
.03 **Statistics**
.04 **Costs and Estimates**
.05 **Contracts and Specifications**
.06 **Drawings**
.07 **Congresses**
.08 **Exhibitions**
.09 **Tests. Laboratories**
501 **Physics and Chemistry of Metals. Metallography**
502 **Alloys**
503 **Ore Dressing. Milling of Ore** (*See also* **under each Metal**)
.1 **Crushing, Grinding etc.**
.2 **Concentration. Sizing. Sorting**
.21 **Flotation**
.3 **Amalgamation**
.4 **Cyanide Process**
504 **Pyro-metallurgy (General)**
.1 **Fuel** (*See also* **222.2, Fuel, under Mechanical Engineering**)
.2 **Refractory Materials**
.3 **Furnaces**
.4 **Gases**
.5 **Smelting**
505 **Hydro-metallurgy (General)**
506 **Electro-metallurgy (General)**
508 **Economics. Trade**
509 **Reports**
510 **Iron and Steel**
.1 **Physics and Chemistry. Metallography**
.2 **Alloys**
511 **Ore Dressing**
513 **Pyro-metallurgy**
.1 **Pig Iron Manufacture**
.3 **Cementation Process**
.4 **Crucible Process**
.5 **Bessemer Process**
.6 **Open-Hearth Process**

514 **Electro-metallurgy**
515 **Foundries and Foundry Practice**
516 **Mechanical Treatment of Iron and Steel**
.1 **Forging**
.2 **Rolling Mills**
517 **Heat Treatment of Iron and Steel**
.1 **Improper Heating of Steel**
.11 **Welding**
.2 **Hardening**
.21 **Case Hardening**
518 **Economics. Iron and Steel Trade**
519 **Descriptions and Reports of Iron and Steel Works**
520 **Gold and Silver**
.1 **Physics and Chemistry**
.2 **Alloys**
521 **Ore Dressing. Milling of Ore**
.1 **Crushing. Grinding**
.2 **Flotation**
.3 **Concentration. Sizing. Sorting**
.4 **Amalgamation**
.5 **Cyanide Process**
522 **Roasting**
523 **Hydro-metallurgy**
524 **Electro-metallurgy**
528 **Economics. Trade**
529 **Reports**
530 **Copper**
.1 **Physics and Chemistry. Metallography**
.2 **Alloys**
531 **Ore Dressing**
.1 **Crushing. Grinding**
.2 **Flotation**
.3 **Concentration. Sizing. Sorting**
.4 **Amalgamation**
.5 **Cyanide Process**
532 **Pyro-metallurgy**
.1 **Roasting**
.2 **Smelting**
.3 **Bessemer Process**
532 **Hydro-metallurgy**
533 **Electro-metallurgy**
538 **Economics. Copper Trade**
539 **Reports**
540 **Lead**
550 **Tin**
560 **Zinc**
580 **Other Metals (Arranged Alphabetically)**
590 **Assaying**

600 **GAS ENGINEERING**
General
.01 **History**
.02 **Laws and Legislation**
.03 **Statistics**
.04 **Costs and Estimates**
.05 **Contracts and Specifications**
.06 **Drawings**
.07 **Congresses**
.08 **Exhibitions**
.09 **Tests. Laboratories**
610 **Natural Gas**
611 **Fields**
612 **Production**
.1 **Wells**
613 **Transmission**
.1 **Pumping Plants**
.2 **Pipe Lines (For Local Distribution, *See* 650)**
620 **Materials for Gas Manufacture**
630 **Manufacture and Works**
631 **Coal Gas**
.1 **Purification**
632 **Water Gas**
.1 **Purification**
633 **Producer Gas**
634 **Coke Oven Gas**

635 **Oil Gas**
636 **Acetylene**
637 **Air Gas**
639 **Other Processes**
640 **Storage**
641 **Station Meters**
642 **Holders**
643 **Analysis and Testing**
650 **Distribution**
651 **Station Governors**
652 **Pressure Instruments and Apparatus**
653 **Pumping Plants**
654 **Flow in Pipes**
655 **Street Mains**
.1 **Valves**
.2 **Subways**
656 **Service Pipes**
657 **Interior Piping and Fittings**
.1 **Regulators**
.2 **Consumers' Meters**
.3 **Piping**
660 **Utilization**
661 **Gas Lighting**
.1 **Illumination**
.11 **Interior**
.12 **Exterior. Streets. Parks, etc.**
.2 **Gas Lamps**
.21 **Burners**
.22 **Incandescent Fixtures**
662 **Gas Heating**
670 **By Products**
671 **Coke**
672 **Tar**
673 **Ammonia**
674 **Sulphur**
675 **Cyanogen**
676 **Naphthalene**
677 **Benzol**
679 **Other Products**
680 **Management**
.1 **Maintenance**
.2 **Rates**
.3 **Finance**
.4 **Valuation**
.5 **Accounting**

700 **CHEMICAL TECHNOLOGY AND MANUFACTURES**
General
.01 **History**
.02 **Laws and Legislation**
.03 **Statistics**
.04 **Costs and Estimates**
.05 **Contracts and Specifications**
.06 **Drawings**
.07 **Congresses**
.08 **Exhibitions**
.09 **Tests. Laboratories**
710 **Chemicals. Dyes. Paints and Varnishes**
711 **Chemicals**
.1 **Chemical Elements**
.2 **Acids**
.21 **Nitric Acid**
.211 **Utilization of Atmospheric Nitrogen**
.22 **Sulphuric Acid**
.3 **Alkalies**
.4 **Salts (For Common Table Salt, *See* 785, Salt, under Foods and Beverages)**
712 **Paint, Varnish, etc.**
713 **Dyes. Colors. Inks**
714 **Distillation Products**
.1 **Wood Distillation**
.2 **Coal Distillation (*See* Gas Engineering)**
715 **Explosives. Matches (*See also* 831, Explosives, under Military and Naval Science)**
716 **Starch, Glucose, Gluten, etc.**

717 **Photographers' Supplies**
719 **Miscellaneous Organic Chemical Industries**
.1 **Vegetable and Animal Oils**
.2 **Soap, Perfumery, etc.**
.3 **Glycerine**
.4 **Candles**
.5 **Waxes**
.6 **Gums and Resins**
.7 **Glue and Gelatine**
.8 **Fertilizers**
720 **Ceramics. Stone, Clay, and Glass. Other Non-Metallic Mineral Products.**
721 **Ceramics**
.1 **Pottery**
.2 Stoneware. **China**
.3 Sanitary **Ware**
.4 **Electrical** Porcelain
.5 **Brick**
.6 **Terra Cotta**
.9 Other Clay **Products**
722 **Glass Manufacture**
723 **Cement, Lime, and Gypsum**
724 **Stone Products. Artificial Stone**
725 **Abrasives**
726 **Asphalt and Other Bitumens**
727 **Petroleum**
.1 Lubricants (*See also* 204, Lubrication, under Mechanical Engineering)
729 **Miscellaneous Non-Metallic Mineral Products**
.1 Asbestos
730 **Metal Manufactures. Machinery. Conveyances**
731 **Gold and Silver Ware. Jewelry**
732 **Iron and Steel Articles** (Not Classified Elsewhere)
733 **Wire Making**
734 **Hardware, Cutlery, and Tools**
735 **Implements. Instruments** (Not Classified Elsewhere)
.1 Typewriters
.2 Cash Registers
.3 Scales
.4 Meters
.5 Watches and Clocks
736 **Machinery.** (Not Classified Elsewhere)
737 **Locksmithing. Gunsmithing**
738 **Vehicles. Conveyances**
.1 Carriages, Wagons, and Sleighs
Automobiles and Motor Trucks (*See* 230)
.2 Bicycles
Manufactures of Miscellaneous Metals (Arranged Alphabetically, *e. g.*, Aluminum, Bronze, Copper, Tin, etc.)
740 **Lumbering. Wood Manufactures**
741 **Logging. Lumbering**
742 **Wood Working. Saw Mills**
743 **Furniture**
.1 House Furniture
.2 Office Furniture
.3 Pianos and Musical Instruments
.4 Mirrors
744 **Cooperage**
745 **Box Making**
746 **Caskets. Undertakers' Supplies**
747 **Pulp and Fiber Goods**
.1 Mats. Baskets
.2 Brooms
.3 Brushes
748 **Cork Industries**
749 **Miscellaneous Wooden Articles**
750 **Paper Making**
760 **Textiles**
.1 Design
.2 Processes
.21 Spinning
.22 Weaving
.23 Dyeing and Finishing
.3 Machinery
761 **Cotton Manufactures**
762 **Woolen Manufactures**

763 **Silk Manufactures**
764 **Flax, Hemp, and Jute**
765 **Dry Goods**
766 **Carpets and Rugs**
767 **Felt**
768 **Cordage and Twine**
769 **Oil Cloth. Window Shades**
770 **Leather Manufactures. Tanning**
771 **Boots and Shoes**
772 **Harnesses. Saddlery**
773 **Trunks. Traveling Goods**
774 **Fur**
779 **Miscellaneous Leather Goods**
780 **Foods and Beverages**
781 **Flour, Feed, and Other Cereal Products**
782 **Sugar and Molasses**
783 **Canning Industries**
784 **Chocolate and Cocoa**
785 **Salt**
786 **Groceries** (Not Otherwise Classified)
787 **Slaughtering. Meat Packing**
788 **Dairy Products**
.1 Milk and Cream
.2 Butter
.3 Cheese
.4 Ice Cream
.5 Casein
789 **Wines and Liquors**
.1 Brewing and Malting. Beer
.2 Wine Making
.3 Distilled Liquors
790 **Miscellaneous Industries** (Arranged Alphabetically, *e. g.*, Celluloid Products, Laundering, Rubber, Tobacco)

800 MILITARY AND NAVAL SCIENCE
General
.01 History
.02 Laws
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.09 Tests. Laboratories
810 **Military Science, General Works**
811 **Strategy and Tactics**
812 **Army Organization and Administration**
820 **Fortifications**
830 **Ordnance (Military and Naval)**
831 **Explosives**
832 **Military Ordnance**
833 **Naval Ordnance**
840 **Naval Architecture. Shipbuilding.**
.1 **Buoyancy and Displacement**
.2 **Resistance**
.3 **Stability**
.4 **Strength of Ships**
.5 **Structural Details**
.51 **Water-tight Compartments**
.6 **Ventilation and Heating**
.8 **Measurement of Ships**
.9 **Miscellaneous Equipment of Ships**
.91 **Steering Apparatus**
.92 **Distillation of Salt Water**
841 **Materials**
842 **Ship Propulsion. Propellers**
.1 **Steam**
.2 **Electric**
843 **Special Types of Vessels** (*See also* 870, **War Vessels, under Naval Science**)
.1 **Ocean Liners**
.2 **River and Lake Boats**
.3 **Vessels for Freight only**
.31 **Oil Steamers. Tank Ships**
.32 **Colliers**
.33 **Ore Ships**

843 **Special Types of Vessels** ***(Continued)***
.4 **Canal Boats. Barges**
.5 **Tugboats**
.6 **Motor Boats and Launches**
.7 **Yachts. Pleasure Boats**
.8 **Survey Boats**
.9 **Other Types**
850 **Yards**
851 **Dry Docks**
860 **Navigation. Shipping**
861 **Nautical Astronomy**
862 **Nautical Instruments**
.1 **Compass**
.11 **Magnetism of Ships and Compass Variation**
863 **Seamanship**
864 **Aids to Navigation**
.1 **Tide Tables**
.2 **Charts**
.3 **Sailing Directions**
.4 **Steam Lanes**
.5 **Signals and other Maritime Regulations. Prevention of Collisions**
.51 **Signals**
.511 **Lighthouses**
.512 **Light Ships**
.513 **Buoys**
.514 **Fog Horns**
.52 **Marking of Ships**
.53 **Flags and Pennants**
.54 **Steering and Sailing Rules**
.55 **Loading Rules**
.6 **Pilotage and Pilots**
865 **Shipwrecks. Accidents**
.1 **Life Saving**
.2 **Salvage**
866 **Shipping. Merchant Marine**
.1 **Directories, Blue Books, etc.**
.2 **Companies**
.3 **Rates**
.4 **Tolls**
.5 **Accounts**
.9 **Inspection**
870 **Naval Science. War Vessels**
871 **Battle Ships and Cruisers**
872 **Torpedo Boats**
873 **Torpedoes**
874 **Submarines**
875 **Armor Plate**
876 **Armament**
880 **Naval Strategy and Tactics**
890 **Naval Organization and Administration**
891 **Registers.**
892 **Navy Customs. Rank of Officers**
893 **Personnel**
.1 **Naval Militia**
.2 **Marines**
894 **Education**
895 **Maintenance and Supplies**
896 **Hygiene**

900 **OTHER SUBJECTS**
Note: For subdivisions, use Dewey Classification.
910 **Philosophy**
920 **Religion**
930 **Sociology**
940 **Philology**
950 **Natural Science**
960 **Useful Arts (Other than Engineering and Manufactures)**
970 **Fine Arts**
980 **Literature**
990 **History**

APPENDIX C

AMENDMENTS TO THE CONSTITUTION, AMERICAN SOCIETY OF CIVIL ENGINEERS, ADOPTED, 1891–1917.

Oct. 3, 1894 ***(Proceedings,*** **Vol. XX, p. 176.)**	**Art. II, Sec. 1, 2. Art. III, Sec. 6, 7. Art. IV, Sec. 1, 2, 3, 7.**	**Abolishes Grade of "Subscribers."**	**Yes, 194 No, 5**
	Art. II, Sec. 8.	**Corporate Members may become Fellows. Previous to this time the grade of Fellows was limited to "contributors to the permanent funds of the Society who may not be eligible for admission as Corporate Members."**	**Yes, 196 No, 3**
	Art. III, Sec. 1.	**Honorary Members "shall be elected only by the unanimous vote of the Board of Direction" instead of "by a unanimous vote of the Board of Direction and such Past-Presidents of the Society as continue to be members of the Society", etc., the Past-Presidents having become members of the Board of Direction on the adoption of the Constitution.**	**Yes, 196 No, 5**
	Art. III, Sec. 2.	**Applicants for membership must be endorsed by written communications from five Corporate Members before applications can be considered by the Board of Direction.**	**Yes, 193 No, 7**
	Art. VI, Sec. 4.	**This Amendment provided that the Secretary be a Corporate Member of the Society and be elected annually by the majority of the whole Board of Direction within twenty days after the Annual Meeting.**	**Yes, 191 No, 6**
Mar. 3, 1895 ***(Proceedings,*** **Vol. XXI, p. 85.)**	**Art. III, Sec. 3.**	**This Amendment required the Preliminary List of applicants to contain the names of "references" for Corporate Members and "endorsers" for Associates, Juniors, and Fellows, in order to make the meaning in Section 2 of this Article clearer.**	**Yes, 265 No, 14**
	Art. III, Sec. 5.	**This Amendment required that ballots on reconsideration of rejected applications for membership must be signed by the voters.**	**Yes, 224 No, 61**
	Art. VII.	**The territory occupied by the Membership was divided into Seven Geographical Districts, thus dividing the non-resident members of the Board of Direction equally among the six non-resident Districts. This amendment increased the elective members of the Nominating Committee from seven to fourteen, who, with the five living Past-Presidents, should nominate officers for the Society; made the term of each member two years; and empowered the Board of Direction to prescribe the mode of procedure in the election of members of the Nominating Committee.**	**Yes, 273 No, 12**
	Art. VI, Sec. 5.	**This Amendment provided for the appointment of an Assistant Secretary by the Board of Direction.**	**Yes, 273 No, 7**

Oct. 6, 1897 (*Proceedings*, Vol. XXIII, p. 164.)	Art. V, Sec. 1.	This Amendment provided that only the "five latest living Past-Presidents who continue to be members" shall be members of the Board of Direction, instead of "all the living Past-Presidents", as previously provided. In the case of the election of Honorary Members, however, all the Past-Presidents shall be members of the Board of Direction.	Yes, 427 No, 47
Oct. 5, 1898 (*Proceedings*, Vol. XXIV, pp. 121, 167.)	Art. VI.	The office of Auditor was abolished and his duties were transferred to the Secretary; provision was made for auditing the accounts of the Society monthly, and the duties of the Finance Committee were widened, in order that the immediate supervision of the financial affairs of the Society might be put into the hands of such Committee.	Yes, 210 No, 1
Oct. 3, 1900 (*Proceedings*, Vol. XXVI, p. 216.)	Art VII, Sec. 2.	The time of appointing the Nominating Committee was changed from the Annual Convention to the Annual Meeting, and the time was fixed for the meeting of such Committee and its presentation to the Board of Direction of the nominations for officers to be elected at the next Annual Meeting.	Yes, 193 No, 53
Mar. 4, 1903 (*Proceedings*, Vol. XXIX, pp. 36, 100.)	Art. III, Sec. 2, 3.	This Amendment provided that hereafter the Board of Direction shall classify the applicant with his consent.	Yes, 401 No, 26
Oct. 7, 1903 (*Proceedings*, Vol. XXIX, pp. 195, 374.)	Art. II, Sec. 6, 7, 8.	By this Amendment provision was made for the omission of the clause, in the case of application for Junior membership, stating that the applicant intends to become or continue to be an engineer.	Yes, 537 No, 49
	Art. III.	By this Amendment all applications are to be sent out as applications for "admission" to the Society without classification into grades; power is given the Board of Direction to transfer persons from a lower to a higher grade; the number of negative votes for exclusion is raised from seven to twenty; and the reconsideration ballot (pink ballot) is abolished.	
Oct. 7, 1908 (*Proceedings*, Vol. XXXIV, p. 408.)	Art. III, Sec. 1, 3, 4.	The election and transfer of applicants in any grade is taken from the membership at large and given to the Board of Direction, the consequent changes in method of election are fixed, and the number of negative votes for exclusion is changed from "20 or more" to "3 or more".	Yes, 892 No, 317
	Art. VI, Sec. 12.	This Amendment confers on the Board of Direction the power of appointing a Special Committee when such appointment is approved by a business meeting of the Society; and, if it is necessary, in the opinion of the Board, that such Committee be appointed in order to accomplish the objects for which its appointment is requested.	Yes, 1123 No, 58
	Art. VII, Sec. 2.	Power is given the Board of Direction to fill any vacancies occurring in the Nominating Committee.	
Mar. 1, 1911 (*Proceedings*, Vol. XXXVII, p. 164.)	Art. IV,	A new Section (13) is added to Article IV by this Amendment, which provides for exemption from dues of Corporate Members and Associates who have reached the age of seventy years, and have paid dues as such for twenty-five years, and also of Corporate Members and Associates who have paid dues as such for thirty-five years.	Yes, 2229 No, 39

Oct. 2, 1912 (*Proceedings*, Vol. XXXVIII, p. 550.)	Art. VII,	By this Amendment the number constituting a quorum at a meeting of the Nominating Committee is fixed at ten; the time of meeting of the Nominating Committee is fixed to take place either at the Annual Convention or not later than July 15; provision is made for the organization of the Nominating Committee; "Official Nominees" and "Nomination by Declaration" are established; nomination by the Board of Direction is provided for, in case the Nominating Committee fails to act; and the time of closing the polls at the Annual Election is changed from noon to 9 A. M.	Yes, 680 No, 31
Mar. 3, 1915 (*Proceedings*, Vol. XLI, p. 150.)	Art. VII, Sec. 1.	The territory occupied by the membership is divided into Thirteen Districts instead of seven, as heretofore.	Yes, 1066 No, 83
	Art. VII, Sec. 2.	This Amendment provides for the method of electing the Nominating Committee from thirteen districts instead of seven.	

AMENDMENTS TO CONSTITUTION, AM. SOC. C. E. REJECTED, 1891-1917.

Mar. 6, 1901 (*Proceedings*, Vol. XXVII, pp. 38, 82.)	Art. II, Sec. 5.	It was proposed by this Amendment to add a clause to Section 5, Article II in order to allow the Board of Direction to transfer any Junior elected prior to the adoption of the Constitution in 1891 to the grade of Associate.	Yes, 282 No, 166
Mar. 5, 1902 (*Proceedings*, Vol. XXVIII, pp. 35, 98.)	Art. III,	This Amendment, if adopted, would have placed the election of all members in the hands of the Board of Direction.	Yes, 343 No, 257
Mar. 6, 1907 (*Proceedings*, Vol. XXXIII, pp. 71, 152.)	Art. II,	It was proposed by this Amendment to raise the standard of membership in the Society by raising the qualifications for admission to the various grades.	Yes, 429 No, 847
	Art. III, Sec. 2.	This Amendment related to applications of engineers not resident in North America and provided that the applicant must possess the necessary qualifications for membership before he is recommended for election to the Society.	
Mar. 3, 1909 (*Proceedings*, Vol. XXXV, p. 160.)	Art. III, Sec. 4.	This Amendment provided that negative votes equal to 1%, or the whole number nearest to 1%, of the total Corporate Membership at the time of voting shall exclude from membership. This Amendment was nullified by the Amendment adopted on Oct. 7th, 1908, and was, therefore, defeated.	Yes, 247 No, 762
Mar. 4, 1914 (*Proceedings*, Vol. XL, p. 176.)	Art. VII, Sec. 1, 2.	(A) By this Amendment, it was proposed to divide the territory occupied by the membership into Thirteen Districts; it also provided for the procedure in appointing the Nominating Committee from such Districts at the Annual Meeting.	Yes, 1494 No, 1628
	Art. VII,	(B) This Amendment also provided for dividing the territory occupied by the membership into Thirteen Districts and the method of procedure of electing the Nominating Committee by ballot to be counted by the Board of Direction and announced to the Annual Meeting.	Yes, 1550 No, 1612

Art. V, Sec. 1, 2.	(C) By this Amendment, it was proposed to change the status of the Secretary by removing him as a member of the Board of Direction. It also defined the terms of officers elected by the Society.	Yes, 1343 No, 1828
Art. VI, Sec. 4, 6.	The changes proposed in this Amendment relate to the method of electing the Secretary by the Board of Direction and would have given the Board of Direction power to determine the salaries to be paid to the Secretary and Treasurer.	

MEMOIRS OF DECEASED MEMBERS

DON JUAN WHITTEMORE, Past-President, Am. Soc. C. E.*

Died July 16th, 1916.

Within the lifetime of Don Juan Whittemore the Engineering Profession progressed from an unrecognized and unorganized state to the first rank among those now established as learned professions. At the time of his birth there were but few broad-minded men who realized the necessity for increasing the facilities for transportation and industrial production in America or who had visions of the advantages to be derived through improvements in machinery and transportation. Some of these few became our first civil engineers. They had foresight and natural ability, courage and perseverance, and were inspired by patriotism and ambition. At the date of his birth, engineering was not a known profession. Those who practised it and became its founders in America were usually self-educated and skilled in some form of manual work, and their knowledge was further increased through acquaintance with each other's efforts. It was a year before his birth that civil engineering was first taught in a school, and it was not until he was twenty-two years of age that the first organization of American engineers was established. This organization, of which Mr. Whittemore was later a most distinguished member, is the American Society of Civil Engineers.

Being actively engaged in engineering work for more than sixty years from 1847, and a close observer, Mr. Whittemore became a potent factor in bringing the Profession of Civil Engineering to its present position of importance and usefulness. He was pre-eminently a railroad engineer, engaged in railroad surveys, construction, and maintenance, and as such did his full share, for a longer period than is granted most engineers, in the development of internal transportation. This is a sufficient field of effort for any one man, and though it occupied his practical activity, it should be recorded that he was always a discriminating student and a patient investigator, and attained an unusual store of knowledge of a wide range of engineering.

A memoir of Mr. Whittemore's life and work would be of historical and professional value if it could be written fully and correctly. Unfortunately for this record, it is found that his peers, who could have supplied the data for such a memoir, have preceded him into that

* Memoir prepared by Charles F. Loweth, M. Am. Soc. C. E., and Onward Bates, Past-President, Am. Soc. C. E., for the American Society of Civil Engineers and the Western Society of Engineers.

country which is obscured from our vision, and it is left to the compilers to enter such fragmentary accounts as are now available, and to supplement them from memory, which does not reach backward to the early part of his working life.

A brief summary of his career, dictated by himself in 1909 for family record, and not revised nor intended for publication, is introduced here, for the reason that his own words, uttered without restraint, will be esteemed of more value than those of another giving the same information:

"I was born in Milton, Vermont, at a little hamlet called Checkerberry Green, December 6th, 1830.

"Parents: Father, Albert Gallatin Whittemore, Lawyer; Mother, Abby Clark Whittemore.

"My first school teachers were Sarah and Lovisa Wright (two giants in height, and mentally strong). Afterward one 'Nerrit', an Irishman and a famous instructor, followed by one Johnson, a collegian, and Dr. F. B. Hathaway, also a superior teacher. At about fourteen years of age my father placed me in school in St. Albans under the tuition of Friar Lawrence, so-called. I also spent a short term in Georgia School. At St. Albans boarded at John Burgess'.

"I then went to Bakersfield, where there was a celebrated school and teachers, the principal being Jacob Spaulding, a famous teacher. I remained there some time.

"Great credit is due my father for his home instruction, he being a natural student, linguist, lawyer, and surveyor. He took great pains in the education, morals, and habits of his children. As I showed a fondness for mathematics and mechanics he secured me a position as surveyor on the Vermont Canada R. R. (in last of 1847), extending from Essex Junction to Rouse's Point, under Phaon Jarrett (a German), Division Engineer, the President of the road being Henry Campbell, of Pennsylvania, at that time called 'Old Whitey', being only forty years of age and white-haired.

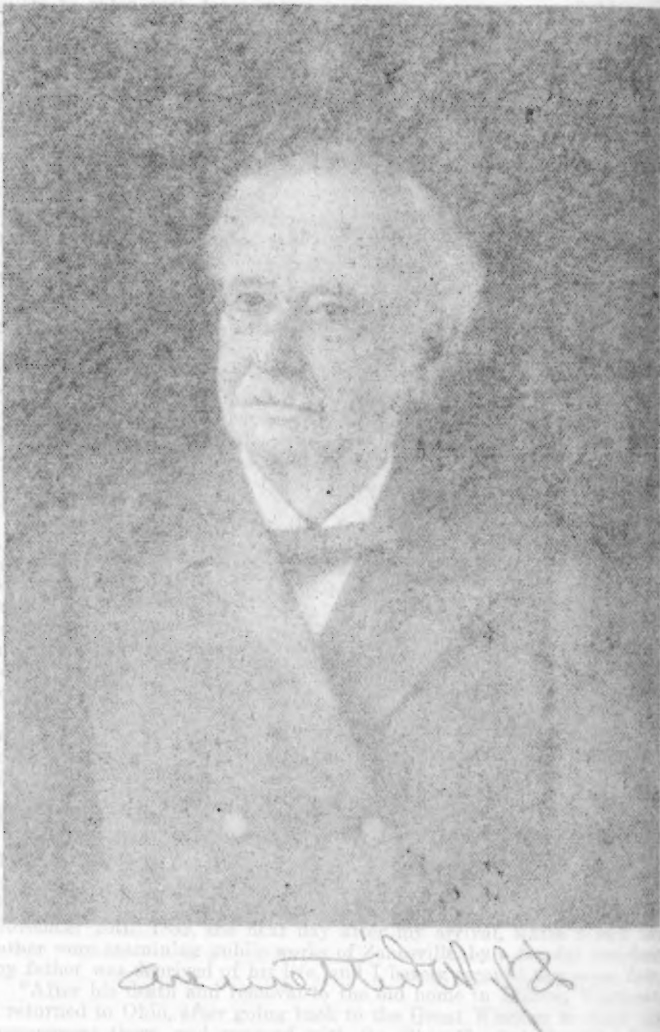
"During this time the first trestle bridge ever built for railroads was erected on Missiquoi Bay between Alburg and Swanton, also a pontoon bridge at Rouse's Point in 1849-50. Was Division Engineer at this time.

"In 1851 went on to the Great Western Railroad, Niagara Falls to Windsor, with position as Resident Engineer.

"In 1852, my father being anxious to have me engage with the Ohio Central, he with others under the firm name of Bradley, Whittemore, Thos. Chittenden and Co., I decided to visit him and see what the prospects of employment were, and whether it was best to go. November 10th, 1852, the next day after my arrival, while I and my father were examining public works of Zanesville, by a fearful accident my father was deprived of his life, and I barely escaped the same fate.

"After his death and removal to the old home in Milton, Vermont, I returned to Ohio, after going back to the Great Western to close my engagement there, and engaged with the Ohio Central Road leading from Wheeling, Virginia, to Columbus—over one hundred miles. My

D J Whittemore



position there was contractor's engineer, representing the estate of my father, the Company then being John Bradley, Norman L. Whittemore, and Thomas Chittenden. I remained there from November to July, 1853.

"This Company took a contract from Milwaukee to Portage City, now a part of the La Crosse Division of the Chicago, Milwaukee and St. Paul Railway, and I was then appointed assistant to the Chief Engineer, Byron Kilbourn, which position I occupied until 1857, when I left and engaged with the same company on the La Crosse and Milwaukee Railroad. Road failed, but company paid the debts—unable to go on.

"From 1857 to 1859 was Chief Engineer of the Southern Minnesota Railroad, running all over the State. This Company failed in 1860, and I went home in the spring.

"In December, 1860, went to Cuba and engaged on the Ferro Carril del Oeste, under Chief Julio Lagbien, as Asst. Chief. Road extended from Havana to Pinar del Rio, western portion of the island. Returned to Vermont in April, 1861.

"Was married in Albany, New York (Brother Clark being present); afterward went back west and became assistant to the Chief Engineer, W. R. Sill, of the La Crosse and Milwaukee until 1863. Soon the Chicago, Milwaukee and St. Paul was organized, when I was appointed Chief Engineer; the first President being Alexander Mitchell. Held this position forty-six years, Russell Sage, Vice-President part of the term. Second President, Roswell Miller; third President, A. J. Earling.

"I conducted examinations through all the passes through the Rocky Mountains. About three hundred men, forty engineers. My present work—extension of the line to the Pacific Coast—the most important of all.

"On the St. Paul Road, east of Butte, Montana, are one hundred and seven miles of bridges, twelve tunnels, the longest one and one-half miles. My part of the work extends from the Missouri River to Butte, Montana, under the name of the Chicago, Milwaukee and Puget Sound Railroad. I now have charge of about nine thousand miles of railroad.

"In 1884 was chosen President of the American Society of Civil Engineers.

"In 1889 went to England to attend C. E. Convention as Past-President. Convention was held in London in Guildhall, a place where no other association had held exercises for two hundred years. Responded to the toast: 'The Civil Engineers'. Met Tyndall,* the scientist, and many other notables, among whom was Sir William Armstrong, inventor of the Armstrong gun.

"After the convention, traveled with my wife, son Eugene, and daughter Fannie, through Switzerland, Germany, Brussels, France, and England.

* Note by Mr. Whittemore's brother: "A pleasant feature of the first trip abroad was the meeting with Professor Tyndall. He gave Don an invitation to bring his family to a luncheon at his home, which was served in a small room or hut among the heather, which he and his wife occupied while building their house. Opposite was Mrs. Humphrey Ward, also near by was Tennyson's home. When they arrived he greeted them very cordially, even throwing his arms about Don, saying, 'This is my friend Whittemore.'—A. G. W."

"Second trip abroad was made in 1903 with my wife and niece Regia, and visited Italy, Switzerland, France, and England.

"On the 50th year of service on the St. Paul Railroad, to commemorate the event, I was, by order of the President, given a private car trip to Mexico and California with everything furnished, to which I invited my wife, and my brother's wife, son, and daughter. A very enjoyable trip without serious accident." "D. J. W."

For about fifty years Mr. Whittemore resided in Milwaukee, Wis., and in a "History of Milwaukee from its First Settlement to the Year 1895" there is found an account of his work and personality, written about 1896, as follows:

"Don J. Whittemore.—In Volume XXI of the *Transactions* of the American Society of Civil Engineers, published some years since, Mr. Whittemore introduced the discussion of an important subject with this unique and strikingly original utterance: 'The Scrap Heap—that inarticulate witness of our blunders, and the sepulchre of our blasted hopes; the best, but most humiliating legacy we are forced to leave to our successors—has always, to me, been brimful of instruction.' The keen analysis of man's mental processes can hardly fail to discover in this utterance of one of the most famous of living civil engineers one of the secrets of his success. While looking forward he has not forgotten to look backward. While planning for the future he has not been unmindful of the past. Delving into the 'scrap heap', he has uncovered mistakes to avoid the repetition of them, has brought to light blunders which he caused to be set up as guide boards pointing out the way not to go thereafter, and has garnered gems of wisdom to crown future efforts. Out of the ashes of failure he has evolved the phœnix of success, and, in overhauling the débris of the scrap heap, it has been a matter of little consequence to him whether the errors exposed to view have been his own or those of some one else, so long as experience had demonstrated that they were errors. Blind dogmatism has had no place in his philosophy, and progress has been the rule of his life.

"To write of Don J. Whittemore all that might properly be written of him in this connection, would be to write an important chapter in the history of western railway construction and development. But the present purpose of the writer is rather to deal with the personality of the man who has attained a celebrity unequalled by that of any other man identified in a similar capacity with western railway enterprises.

"Born in Milton, Vt., December 6th, 1830, Mr. Whittemore is a descendant, seven generations removed, of Thomas Whittemore, who was one of the earliest settlers of Charlestown, Mass. Thomas Whittemore came to this country from Hitchin, an ancient market town in Hertfordshire, near London, about the year 1640, and settled in that part of Malden which is now Everett, Mass. In 1645 he was the owner of a farm on the western border of Chelsea, which remained in the possession of his descendants until 1845, a period of two hundred years. Albert Gallatin Whittemore married Abby Clark, also of English ancestry, and Don J. Whittemore was the second son born

of this union. The elder Whittemore was a noted lawyer of Milton, Vt., participated as a volunteer in the battle of Plattsburg in 1814, was distinguished locally as a fluent and impressive public speaker and linguist, and a thorough mathematician. The son received his early education under the preceptorship of his father, in whom he had a most competent teacher, and later attended for a time Bakersfield Academy. Leaving school when he was seventeen years of age, he became connected with the engineering corps of the Vermont and Canada Railroad Company, and his proficiency in the science of civil engineering, even at that early age, is attested by the fact that when he was nineteen years old he was appointed Assistant Engineer of this company, having charge of construction of the line between Swanton, Vt., and Rouse's Point, N. Y. Having completed this work, he was appointed Assistant Engineer and placed in charge of construction of a division of the Great Western Railway of Canada. He retained that position until 1852, when the sudden death of his father brought about a change of his relations. The elder Whittemore was at that time largely interested in the building of the Central Ohio Railway, between Zanesville, Ohio, and Wheeling, Va. He was accidentally killed while inspecting the superstructure of a bridge across the Muskingum River at Zanesville, and the responsibility of looking after his interests devolved upon the son, who happened to be paying him a visit at the time of his death. Resigning his position with the Great Western Railway Company, D. J. Whittemore became contractor's engineer on the Central Ohio Railroad, and retained that position while giving attention to the adjustment of his father's affairs.

"In this way he became interested in what was looked upon in those days as Western railway building, and in 1853 was transferred to the field of his future activity and enterprise in the Northwest. He was appointed that year assistant to the Chief Engineer of the La Crosse and Milwaukee Railroad Company, then in process of construction. At the end of four years in this service he resigned his position with the La Crosse and Milwaukee Road to become Chief Engineer and Director of the Southern Minnesota Railroad Company, locating about two hundred and fifty miles of that Company's line within the next two years. In 1859 work upon that line of railway was suspended, and, broken in health by the hardships which he had endured in traversing a country then in a condition of primitive wilderness, Mr. Whittemore went to Cuba, where he accepted the position of Assistant Chief Engineer on the Ferro Carril del Oeste (Western Railroad of Cuba) with which he was connected nearly a year.

"Returning to Wisconsin in 1860, he again became Assistant Chief Engineer of the La Crosse and Milwaukee Railroad Company, continuing his connection with that company until 1864, when its line was merged into the Chicago, Milwaukee and St. Paul Railway System. With this great corporation, which now owns and operates over six thousand miles of railway, he entered upon a term of service, as Chief Engineer, which has extended over a period of forty years, during which time the company has developed one of the great railway systems of the world.

"In the midst of his exacting railway duties he has found time not only for general scientific research, but for special pursuits which have led up to important developments.

"In 1874, or possibly a little before that time, his attention was called to the hydraulic features of the rock deposits underlying a portion of this city [Milwaukee]. He began a series of experiments which developed the fact that a first-class hydraulic cement could be made from the rock. The result was the formation of the Milwaukee Hydraulic Cement Company, in which he became interested as a shareholder, and the establishment of a plant which now sends to the market upward of 500 000 barrels of cement every year. For many years he was a director of this company, but in 1891 he resigned this directorship to become Vice-President of the Western Portland Cement Company of Yankton, S. Dak., an enterprise of which he was also one of the founders.

"In 1884 he was honored by the American Society of Civil Engineers with the Presidency of that Society, and the University of Vermont, his native State, has conferred upon him the degree of Civil Engineer; while the University of Wisconsin, his adopted State, has recognized his scientific attainments by conferring upon him the degree of Doctor of Philosophy. In the American Society of Civil Engineers he has wielded an important influence for many years, while he has also been conspicuously identified with the American Society of Mechanical Engineers, the Western Society of Engineers, and honored with a membership in the Institution of Civil Engineers of England.

"In 1889, when a delegation of about two hundred and fifty of the civil, mechanical, and mining engineers of America visited England, France, and Germany, Mr. Whittemore was made honorary chairman of the delegation, and was the recipient of distinguished honors at the hands of the engineers and scientists of the Old World. Among the pronounced scientists with whom he became intimately acquainted on that occasion was Professor Tyndall, and a friendship sprang up between the two men which resulted in a correspondence kept up until Professor Tyndall died. As Vice-Chairman of the General Committee of the World's Congress Auxiliary to the Columbia Exposition, having in charge the conduct of the World's Congress of Engineers, held at Chicago in 1893, Mr. Whittemore had an opportunity to reciprocate the courtesies extended to him some years earlier while abroad, and he was a prominent participant in the deliberations of that famous gathering of engineers. A pleasing and ready writer, he has been a frequent contributor to the published transactions of the American Society of Civil Engineers, and now and then engaged to some extent in the discussion of important engineering problems through the newspaper press."

Among Mr. Whittemore's chief characteristics was an unusually retentive memory. He has been accused of remembering the exact location of every slope stake placed by him in the early part of his career, when he was an instrument man. This, of course, is an exaggeration, but that quality of his mind is shown by the following item

from the *Burlington Free Press and Times* of April 29th, 1887, printed forty years after the occurrence described:

"VERMONT AND CANADA ROAD

"Where its Construction Was First Begun—

"An Interesting Letter.

"We are permitted to copy the following from a letter written by D. J. Whittemore, Chief Engineer of the Chicago, Milwaukee and St. Paul to his brother, A. G. Whittemore, of this city.

"'I notice in Rann's history of Chittenden County, page 191, it is stated that the work of constructing the Vermont and Canada railroad was begun early in September, 1848, in the northern part of Georgia. If the author means to convey the impression that work was first begun there I feel sure that he is in error. The first ground broken in the construction of that line was at near the north end of the Y at Essex Junction. There were present at this ceremony Charles Paine, President; H. R. Campbell, Chief Engineer; Phaon Jarrett, Assistant Chief Engineer; Ambrose Pierson, First Assistant Engineer; ———— Bushnell, Second Assistant Engineer; D. J. Whittemore, Rodman, and several citizens of Essex. After a few impressive remarks on the importance of the work by President Paine, he loaded one barrow with earth and wheeled it into embankment. Each of the others in the order named did the same, after which all adjourned to the office near by and drank a bottle of champagne. The shovel and wheelbarrow used were appropriately inscribed with the names of all the actors, in Capt. Jarrett's best style of lettering, and I believe were sent to Northfield and presume were burned in the conflagration which destroyed the Vermont Central buildings three years afterward. It may be that I am now the only one living who participated in this event. This is of very little importance, anyway, except to correct the impression conveyed in the work referred to.'"

In Mr. Whittemore's service of more than half a century with the Chicago, Milwaukee, and St. Paul Railway, he encountered almost every problem of railway engineering, and was able to collaborate with those who had made particular study of special problems and to learn and apply the knowledge acquired by their intensive experience, to their mutual satisfaction. This was especially the case in the matter of bridges, when, with the aid of such men as C. Shaler Smith, Moritz Lassig, Members, Am. Soc. C. E., and others, the structures over the rivers crossed by his lines were of such character as to make them noted examples of correct and bold construction. A few such bridges may be mentioned here, for the reason that they were constructed at early dates, when long-span bridges were rare, and builders were feeling their way toward present achievements. Kilbourne Bridge, over the Wisconsin River, a wooden Howe truss structure with a principal span of 242 ft., carrying a highway on its lower chords and a railway on its upper deck, served its purpose and was replaced in 1887 by an

iron structure, which in turn was replaced by a steel bridge a few years ago. Moritz Lassig was the builder of the wooden bridge, C. Shaler Smith of the iron bridge, and the steel bridge was built by the Railway Company some years ago.

Sabula Bridge, an iron structure over the Mississippi River, was built by C. Shaler Smith about 1879. This bridge was at that date considered one of the best examples of such construction, and it was selected by Professor Malverd A. Howe, M. Am. Soc. C. E., as an example for use in university instruction, and was made by him the subject of a textbook. This bridge was examined by experts, a few years ago, who pronounced it of excellent design and well maintained, but was recommended for replacement to provide for increased live loads from modern rolling stock. This recommendation has since been put into effect.

Minnehaha Bridge, another iron structure built by Mr. Smith, over the Mississippi River at Fort Snelling, Minn., finished in 1880, was a deck bridge, with a central span of 324 ft., two flanking spans of 270 ft. each, and some approach spans, with a height of 108 ft. from high water to base of rail. It was similar in design to Mr. Smith's cantilever bridge over the Kentucky River, which preceded it, and was considered a distinct advance in bridge construction, attracting favorable comment from bridge engineers. The Minnehaha Bridge was replaced in 1901 by a double-track steel bridge.

These bridges, though not comparable in size with the great ones of recent years, are worthy of record as advancements in the art of bridge engineering as it was thirty and more years ago. Mr. Whittemore's specifications for iron bridges, written in collaboration with the bridge engineers of those days, were classics in their line, leading the art of the time in form, completeness, and correctness of theory and design.

Mention has already been made of Mr. Whittemore's connection with a pontoon bridge at Rouse's Point in 1849-50, when he was scarcely more than a boy. It is not strange, therefore, that when, some years later, it was necessary to cross the "Father of Waters" at Prairie du Chien, he accepted the proposal of the Hon. Thomas Lawler of that city to cross both channels of the river with pontoons. For more than forty years these bridges, with large openings for navigation, have been maintained safely and economically, and have been copied at other locations.

In Mr. Whittemore's practice he had great experience with general railway contractors, and employed many of the most notable of them. Nearly all these men whose aid he secured are now dead. If they were living, material for this memoir could be gathered from them which would fill a volume of value to the Profession. He commanded

the respect of such aids, and it is a notable fact that final settlements of contracts, with remarkably few exceptions, were made to the satisfaction of both parties, who accepted his rulings as just and fair. This fact, which will be appreciated by engineers of experience, was so well known that it was frequently commented on.

The diversity of work coming under his supervision was so great and the period of his office was so long that it is impossible to bring the details of it within the scope of this memoir. Mention should be made, however, of the careful and complete work which he did with his own hands. Specifications and bills of material made out in his own handwriting, which have been seen by the compilers of this memoir, are so plain and yet so complete that there is no question raised as to their exact meaning.

Mr. Whittemore was more than a railway engineer. He was a student and investigator in many lines and fields, with a rare mind capable of seeing through to the end, and he employed his facilities in his leisure moments on a multiplicity of interests. Those who were with him at such times would be surprised at the questions he would ask and at the diversity of their character, and would be instructed by his sage remarks covering a wide range of knowledge. He was on such occasions a source of instruction and an inspiration for younger men. On business trips over the lines of his company, while not in the least neglecting the work immediately in hand, he would give his assistants problems which had no immediate relation to present work but were tests of their knowledge and of their capacity for considering new questions. This seemed to be a lifetime habit in educating himself, and one having close relations with him would observe with surprise the lines of study to which he applied himself, because they were foreign to the ordinary routine of his business. He carried with him this habit of study and reflection, and friends visiting him in his quiet hours would find him absorbed in Isaac Newton's "Principia", or in some abstruse problems more or less remotely connected with those encountered in his usual practice.

He possessed an analytical mind, with a natural talent for research, and in the pursuit of knowledge he made in a quiet way valuable contributions to practical science. It was characteristic of him to be thorough in analysis and in experimentation, and always interested in the subject under consideration. Mention has already been made of his interest in the production of hydraulic cements and his successful establishment of that industry in the City of Milwaukee. This was followed by similar work resulting in the successful commercial manufacture of Portland cement at Yankton, S. Dak., at a time when Portland cements were nearly all imported and none was made west of the Alleghenies. It is not possible to place a tangible value on his work as a cement investigator, but that it was of very great value in the

development of a large section of our country is evident when considered in its relation to the magnitude of the cement industry of the present time.

He gave much attention to the production of aluminum at a period antedating its manufacture on a commercial scale, and though successful in extracting it, did not reach the achievement obtained by the method at present in use.

His investigations covered a wide range of subjects, and, as might have been expected, some of his work resulted more in personal satisfaction than in practical value. An instance of this is found in the "Equilibristat" invented by him, being an instrument to be mounted in a railway car to indicate whether the super-elevation of the outer rail on curves is proper for the speed of the car passing over it. He exhibited an equilibristat and read a paper on it before the Western Society of Engineers.

Mr. Whittemore's loyalty to the railway of which he was Chief Engineer led him to consider the interest of his employer as his first and principal duty, and under that obligation he found little time to devote to personal interests outside of those pertaining to his office. Mention has been made herein of his interests in Milwaukee and Yankton cements, and a large user of both of these cements testifies that he was unable to get Mr. Whittemore to recommend either of them, even when the question was pressed for an answer. As a consequence of this policy he did not participate to any great extent in employment as a consulting engineer, and not at all without the approval of his superior officers. He might have done more of such work without detriment to his employer and to the advantage of the Profession if he had not had so scrupulous a regard for his fixed obligations. His long experience, preserved by a retentive memory, with his judicial faculty of mind, was a valuable professional asset which ought to have been used more frequently for the general benefit. There were occasions when he had to yield to imperative demands for his knowledge and judgment. On two different occasions, when disastrous accidents occurred to the bridge over the Missouri River at St. Charles, Mr. Whittemore was summoned to investigate and determine the causes of failure. No record has been preserved of his professional services aside from those rendered in his official position, nor of the requests for such services which he felt he ought not to accept. In 1888, he and the late Joseph M. Wilson and A. P. Boller, Members, Am. Soc. C. E., were appointed by the City of Providence, R. I., as a commission to investigate an important city problem, and much credit was given the commission for its report. In his long period of active practice there was without doubt much which could profitably be referred to in this memoir if the facts were obtainable.

Mr. Whittemore was a clear thinker, with facility for expression as a writer and speaker. With a modest man, as he was, such traits lie dormant until developed and brought to light by circumstances. When it became known that he was a well-informed and pleasing speaker, opportunities for speaking were so frequent that he acquired the habit of declining, and it was a rare occasion when he could be induced to make an address. He was urged by friends to write contributions to the history of engineering in America, which, with his long experience and observation, he was well qualified to do, but he would not yield to their solicitations. His habit of reticence seemed to grow upon him with advancing age, and it is much to be regretted that, when he began to lay down his routine duties, he did not contribute from his ripe experience for the benefit of those who had not acquired that which they knew he possessed. He departed this life after outliving all those with whom he worked in the "early days", and these men left mostly memories and traditions without actual records, but their work is not wholly lost, for those whom they educated will pass their knowledge on through their successors, and what they accomplished while living remains an intangible asset for those who come after them.

Mr. Whittemore was by nature more an engineer than an administrator. If his administration duties had outweighed his others, it is probable that he would not have limited his energies to the practice of engineering. He was associated with railway construction and maintenance throughout a lifetime, covering the development of railroads from almost their beginning, and, if his ambition had been in the line of railway promotion and management, it is altogether likely that he would have become associated with them as business enterprises rather than as fields for engineering effort. In other words, his work was professional instead of commercial in character. He must have had many opportunities for profitable employment as a contractor or a promoter, and it is characteristic of him that he appreciated his office of Chief Engineer and resisted temptation offered by more lucrative employment.

He understood the requirements of an engineering organization, and his ability to discern character enabled him to gather about him efficient and loyal assistants and to employ competent contractors. His relations with all such were friendly, and promoted the feeling of common interest instead of antagonism. As indicative of his character and of the esteem in which he was held by his associates, the following extracts from letters, sent to the compilers, may be properly introduced in this memoir.

From Mr. E. O. Reeder, Assistant Chief Engineer, Chicago, Milwaukee and St. Paul Railway:

"Although I served under and was very closely associated with D. J. Whittemore, as a subordinate, for the greater part of the time from early in 1875 until his retirement from active service in December, 1910, I find some difficulty in suggesting matter for his memoir.

"Few engineers, or in fact few managing or directing officials of a railway company, have a record of such a close and direct connection from the beginning, with the building up of a great railway of the magnitude of the Chicago, Milwaukee and St. Paul Railway.

"In the early 'Fifties', practically at the beginning of railway construction west of the State of Ohio, Mr. Whittemore was engaged on the surveys and construction of lines in Wisconsin, which were the nucleus of and are still a part of the C., M. & St. P. Railway System. I have heard him remark that he once surveyed a line across the site of the present heart of the City of Minneapolis and that no house or building was then there to obstruct his work.

"From these early days until his death, except for a few short intervals, he was connected with our company, and for the greater part of this period he was the head of the Engineering Department, and as such he planned, directed, and supervised the construction of some 6 000 miles of new railway now forming a part of the great system of over 10 000 miles, and he also planned and directed the many works and improvements that were undertaken in developing the system.

"He held to a marked degree the confidence of the managing officers of the company, from S. S. Merrill, who, perhaps more than any other, laid the foundation of the system, to A. J. Earling, who extended and built it up.

"Mr. Whittemore's reputation for justness and fairness as between the Railway Company and its contractors was so high that his decision as arbitrator was seldom, if ever, questioned, and because of this high estimation the Railway Company was able to contract its work to the best advantage, and few, if any, controversies resulted in lawsuits in settlement of contracts.

"Mr. Whittemore's personality, character, and high mental attainments commanded the admiration and respect of all who knew him. He possessed a strong and active mind.

"His subordinates loved and respected him, and felt complete confidence that he would be just and loyal in his treatment of them.

"To me, among his most marked characteristics, were his power of concentrating his mind on and his ability to thoroughly analyze a subject. Physically, he was not very rugged, and consequently had to depend upon observations and reports of others for his knowledge of many operations and conditions, especially as regards examinations and explorations for new lines. He had, to a remarkable degree, the faculty of getting from reports of others a more complete insight and understanding of pertinent facts and conditions than an ordinary man would get from actual observation, and of thereafter retaining these facts in mind.

"His reputation as an engineer can best be commented on by some one who has a wider knowledge of it than I have. His contemporaries

and associates in the profession were of the most prominent engineers of the country and of the world, and his rank among them was high."

From Mr. A. G. Baker, Assistant Engineer, Chicago, Milwaukee and St. Paul Railway:

"I have to say that the following few lines relative to my personal estimate of Mr. Whittemore are submitted with diffidence, as the subject is difficult to handle as it deserves. Having been intimately associated with Mr. Whittemore for twenty-five of the thirty-eight years of service with the Chicago, Milwaukee and St. Paul Railway, I venture to say that it is possible for me to form a fairly true estimate of his qualities as a man and engineer.

"As a man and chief I think there were few, occupying the position that he did, who had the quality of winning the respect and loyalty of his subordinates, and, in my case, a degree of affection hard to express.

"On his part, he was loyal to his assistants who were deserving and tried to do their duty. In this respect, one of his chief characteristics was the solicitude he displayed for the comfort and safety of any one of his engineers sent on a specially hazardous undertaking in the line of reconnaissance or survey. Many instances of his thoughtful consideration along these lines occur to the writer, and others will bear witness to the same.

"Another characteristic was his absolute fairness in dealing with contractors. His instructions were explicit to all engineers on construction work to treat all contractors, from principals to sub-contractors and 'station men', with due consideration. All the leading contractors in the West with whom I have had business relations concur in the general statement that they could safely leave all matters of classification, etc., to Mr. Whittemore, feeling sure they would receive just treatment.

"As to his qualities as an engineer, it would seem superfluous for the writer to enlarge on his abilities, too well known to require endorsement from this source. However, it might not be out of place to mention one quality so often noticed, *i. e.*, his ability to grasp difficult situations relative to location of lines, from reports and examination of maps and profiles, without having a personal knowledge of the country traversed, and to make valuable suggestions as to the betterment of conditions. This quality was also observed in matters relating to construction. This point was called to the attention of the writer more particularly during the construction of the Puget Sound Extension. Mr. Whittemore had not been able to go over the new line until the track was laid and the line was in operation. In 1910 it was the privilege of the several Engineers of Construction to accompany him in a business car over their portions of the work. While watching the line from the car he would frequently notice and call attention to places where particular difficulties occurred and were referred to him for advice and instruction; he having recognized them from his familiarity with the profile and map of such localities. His memory and 'bump of location' were extraordinary.

"In his dealings with his assistants, relative to their work, his fairness was proverbial. Where necessary, his criticisms were severe, and he seldom failed to strike at the root of faults. In conclusion, I cannot refrain from stating that Mr. Whittemore was as much a father to me as a chief during the many years I served under him, and my feelings of affection for him were strong."

Mr. Whittemore was a charming social companion—with rare humor he could entertain his friends for an evening and make it an occasion to be long remembered. He was a good story teller, and his reminiscences, based on the experiences of his early practice, were fascinating to younger engineers. His disposition was naturally a retiring one, and his friends greatly regretted that in the latter years of his life he seemed to avoid the gatherings of engineers. At the last Convention of the American Society of Civil Engineers attended by him (held in Chicago in 1910) he told a friend, who met him at the hotel designated as Convention Headquarters, that he was too old, that he did not know those whom he was to meet, and that they would not care to meet him. This friend arose to the occasion and privately repeated what he had said, with the result that he was given an ovation by those of all ages who were present and for a short time was made to feel young again. The present generation will have to disappear before personal recollections of him will cease to be expressed.

He had a kind heart, and, like many others who have that possession, he did not display it in public. Always helpful of others, his benefactions were not recorded. All his friends will remember him with warm feelings of regard and respect, and will continue to speak of him as one of the early builders of our profession.

On December 6th, 1910, Mr. Whittemore retired from the office of Chief Engineer of the Chicago, Milwaukee and St. Paul Railway Company, and did not thereafter perform any active duties, although he carried the honorable title of "Consulting Engineer" until the date of his death. He is survived by his widow and his daughter, who is the wife of Philip N. Littell, author and publisher, residing in New York.

Mr. Whittemore's interest in the advancement of his profession was shown by his active support of its societies in contributing to their transactions and in his labors on their committees. The Western Society of Engineers, in which he held membership, honored him by election as Honorary Member. He was also a Member of the Institution of Civil Engineers of Great Britain.

Mr. Whittemore was elected a Member of the American Society of Civil Engineers on July 10th, 1872, and an Honorary Member on January 6th, 1911. He served as a Director in 1881 and as President in 1884.

DANIEL MARSHALL ANDREWS, M. Am. Soc. C. E.*

Died June 28th, 1917.

Daniel Marshall Andrews was born in Americus, Ga., on October 24th, 1853. He was the son of Judge Garnett Andrews and Annulet Ball Andrews, of Washington, Ga.

In the spring of 1872 he entered the Engineering School of Georgia University, at Athens, Ga., and would have been graduated in 1874, had not a serious illness prevented him from taking the final examinations. In later years his Alma Mater issued his diploma and conferred on him the degree of C. E., as of the class of 1874. This action was based on his excellent record as a student, and his subsequent high standing in the profession of Civil Engineering.

Mr. Andrews left college in the midst of the panic of the Seventies, and, after trying unsuccessfully for six years to grow 5-cent cotton at a profit, he, in 1881, returned to his profession.

From 1881 to 1884 he was employed on the survey, location, and construction of railroads in Georgia and South Carolina, embracing certain lines of the Seaboard Air Line System in these States. From 1884 until his death he was engaged in river improvements with the United States Engineer Department, except for a short time in 1902 when he investigated and prepared estimates of cost for the Galveston sea wall. While in the Government service, Mr. Andrews' work consisted of the design and construction of navigation locks and dams, the regulation of rivers, the investigation of many Southern rivers, together with the storage reservoirs on their headwaters, with a view to the co-ordination of power and navigation, and, at the time of his death, he had charge of the improvement of the Coosa River and its tributaries in Georgia and Alabama, the Alabama River in Alabama, the Chattahoochee River in Georgia and Alabama, the Flint River in Georgia, and the Apalachicola and Choctawahatchee Rivers in Alabama and Florida.

He was a member of the following scientific and technical societies: The American Association for the Advancement of Science; the American Historical Association; the International Navigation Congresses; the National Geographic Society; the Alabama Anthropological Society; the Bartram Society of Natural History; and the Alabama Technical Association.

The following is a partial list of technical and historical papers written by Mr. Andrews: "The Economic Improvement of the Coosa

* Memoir prepared by B. M. Hall, M. Am. Soc. C. E.

and Alabama Rivers in Georgia and Alabama",* "Backwater Above Dams",† "Foundations in the Coosa and Black Warrior Rivers",‡ "Dam No. 5, Coosa River, The Problems of Location and Construction",‡ "The Improvement of Non-Torrential Streams",‡ and "The Route of DeSoto from Cofitachequi in Georgia to Coosa in Alabama",§ read before the Alabama Anthropological Society.

Although Mr. Andrews was a U. S. Assistant Engineer for the greater part of his life, engaged in engineering works of considerable magnitude, he was also a great student of science and general literature.

On April 20th, 1897, he was married to Miss Adeline Van Court, of Natchez, Miss., who survives him, together with their three sons, Daniel Marshall, Jr., Van Court, and Garnett.

At the time of his death, and for many years previous, his home was at Montgomery, Ala. In June, 1917, he went to Chicago, Ill., for an operation on his throat, and died there on the 28th of that month.

Mr. Andrews, known familiarly by his boyhood friends as "Marsh" Andrews and by the general public as Major Andrews, was a Christian gentleman, a man of integrity and honor, and an able engineer of scholarly attainments. He was a man of positive convictions, dignified bearing, and a delightful personality. He was devoted to his home, his family, and his friends and was enthusiastic in his professional work.

He was elected a Member of the American Society of Civil Engineers on March 2d, 1892, and always took an active interest in its affairs.

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* *Transactions*, Am. Soc. C. E., Vol. L, p. 363.

† *Engineering News.*

‡ *Professional Memoirs*, Corps of Engrs., U. S. A.

§ *American Anthropologist*, Jan.–March, 1917.

WILLIAM SINCLAIR BACOT, M. Am. Soc. C. E.*

DIED OCTOBER 31ST, 1917.

William Sinclair Bacot, the son of Robert Cochran Bacot and Mary Gilchrist Bacot, was born in East Orange, N. J., on April 19th, 1860. He was a direct descendant of Pierre Bacot, a French refugee who fled to America after the Revocation of the Edict of Nantes and settled in Charleston, S. C., in 1685.

Mr. Bacot was prepared for college at Hasbrouck's Institute, in Jersey City, N. J., and entered Princeton College in 1877, from which he was graduated with the class of 1881, with the degree of C. E.

After leaving college he was first employed as an Assistant Engineer on the construction of the Hackensack Water Company's Works in New Jersey. From 1882 to 1908, he was engaged in private practice, having an office in New York City and making a specialty of improved highway building and hydraulic engineering and construction.

Mr. Bacot was appointed by the United States Government to build the Good Roads Exhibit at the World's Fair at Chicago. For several years he was County Engineer of Staten Island, New York, and built good roads in that place and also at Lenox, Mass., and Burlington, Vt.

Among the water-works constructed and operated by Mr. Bacot were those at Fishkill, Portchester, Matteawan, and Mount Vernon, in New York State, and Greenwich, Conn. His last work was the development of the water-works for the City of Utica, and the surrounding towns and villages, covering the period from 1900 to the present time. He designed and built the West Canada Creek supply system for Utica, and welded into one the water-works of Utica, New Hartford, Whitestown, Oriskany, and other adjoining towns and villages.

At the time of his death, Mr. Bacot was President of the Consolidated Water Company of Utica, N. Y., and its Chief Engineer, positions which he had held almost continuously for many years.

He was of a retiring disposition, although genial and of a lively and enthusiastic temperament, inheriting the latter characteristics from his French ancestry.

In May, 1917, he was married to Anne Harrison Hendryx, of Cincinnati, Ohio, a great-granddaughter of the late President William Henry Harrison.

He is survived by his brothers, John V. and Richard Wainwright Bacot, and a sister, Mrs. Annie B. Roundey, and several nieces and nephews.

Mr. Bacot was elected a Member of the American Society of Civil Engineers on October 1st, 1890.

* Memoir prepared by John V. Bacot, Esq., Utica, N. Y.

VAN BRUNT BERGEN, M. Am. Soc. C. E.*

Died April 27th, 1917.

Van Brunt Bergen was born at Bay Ridge, N. Y., on April 29th, 1841. He was a son of the Hon. Teunis G. Bergen and Elizabeth Van Brunt Bergen. He was graduated from the Brooklyn Collegiate and Polytechnic Institute in 1859, and in 1860 entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1863, with the degree of Civil Engineer.

In 1864, Mr. Bergen was appointed Rodman on the Brooklyn Water Works, and thereafter served in various capacities in the Department of City Works under Chief Engineers Kirkwood, Lane, and Adams. On the resignation of the latter, in the early Seventies, he became First Assistant to Chief Engineer Robert Van Buren, M. Am. Soc. C. E., and continued in that capacity until the resignation of Mr. Van Buren in 1893. Mr. Bergen was then appointed Chief Engineer, and as such served under the administration of Mayor Schieren, with Mr. Alfred T. White as Commissioner of City Works, for a period of about two years. Mr. Bergen then retired from public service and devoted most of his time to the management of his large real estate holdings in Brooklyn.

Mr. Bergen's principal engineering work, while in the service of the City of Brooklyn, was the laying of the 48-in. water main from the Ridgewood Reservoir to the corner of Clinton and Atlantic Avenues; the construction of quite a number of the city's sewers; the development of the city's water supply in the valley of the Hempstead stream; and much other work connected with the water supply of the city.

In 1884, he compiled a short account of "The Department of City Works, Water and Sewerage," which was printed in "Stiles' History of the County of Kings and the City of Brooklyn, New York."

Mr. Bergen was a member of the following clubs and societies: The Engineers Club of New York, the Hamilton and Crescent Athletic Clubs of Brooklyn, the Holland, St. Nicholas, and the Long Island Historical Societies, and also of many scientific associations.

On August 3d, 1871, he was married to Elizabeth Emma van der Veer, daughter of Cornelius van der Veer, of Somerville, N. J. One child, a son, Henry van der Veer Bergen, was born on August 8th, 1873.

* Memoir prepared by Gustave Kaufman, M. Am. Soc. C. E.

After his retirement from active engineering work, Mr. Bergen traveled extensively in the United States, Canada, Mexico, the Levant, and Egypt.

He was a quiet and modest man, of very high attainments. He had a very kind disposition, and was held in high esteem by all who knew him.

Mr. Bergen was elected a Member of the American Society of Civil Engineers on June 17th, 1868.

HERBERT JAMES CHAMBERS, M. Am. Soc. C. E.*

Died May 20th, 1918.

Herbert James Chambers, the only son of James and Fanny (Flood) Chambers, was born in Brookfield, Mass., on November 7th, 1872. His education was received at the Brookfield Public Schools, including the High School, and at the Worcester Polytechnic Institute, from which he was graduated in 1895.

From July, 1895, to August, 1900, Mr. Chambers was employed as Draftsman and Engineer by several New England companies, among which were the Springfield Construction Company and the Berlin Iron Bridge Company.

In 1900, he came to New York City, and, in August of that year, entered the employ of Milliken Brothers, which firm was engaged in engineering and construction work in foreign countries. In 1902, he was promoted to the position of Contracting Engineer, which place he retained until July, 1907, when he resigned to enter into partnership with John W. Hamilton, M. Am. Soc. C. E., as a member of the firm of Hamilton and Chambers. This partnership which was organized for the purpose of conducting an engineering and contracting business, was eminently successful, and, in 1916, was incorporated as the Hamilton and Chambers Company, Incorporated. At the time of its re-organization, Mr. Chambers became the Treasurer of the Company, which office he held until his death, at his home in Mount Vernon, N. Y., on May 20th, 1918.

He was a man of strong character and had a high standard of conduct which impressed all who came in contact with him both in business and social relations. He was an intelligent, energetic and capable engineer, with a strong grasp of detail, which enabled him to prosecute his work most successfully, and he has left a record of probity with the men with whom he was associated, which, without question, will influence their conduct of life.

In 1900, Mr. Chambers was married to Miss Marion French, of Attleboro, Mass., who, with a daughter, Adelaide French Chambers, survives him.

He was a member of the National Geographic Society, the Sewanoy Country Club, the Whitehall Club, and the Engineers Club of New York City.

Mr. Chambers was elected an Associate Member of the American Society of Civil Engineers on November 5th, 1902, and a Member on January 3d, 1911.

* Memoir prepared by J. W. Hamilton, M. Am. Soc. C. E.

HIRAM MARTIN CHITTENDEN, M. Am. Soc. C. E.*

DIED OCTOBER 9TH, 1917.

Hiram Martin Chittenden, the son of William F. and Mary Jane (Wheeler) Chittenden, was born at Yorkshire, N. Y., on October 25th, 1858. He was graduated from the United States Military Academy at West Point, N. Y., in 1884 and assigned to the Corps of Engineers.

After three years at the Engineer School of Application at Willetts Point, Lieut. Chittenden was graduated and sent to Omaha, Nebr., as Engineer Officer of the Department of the Platte. He held this appointment for two years, during which time he prepared a topographical map embracing Colorado, Wyoming, and Utah, and portions of the surrounding States.

In 1889, he was assigned by the War Department to take charge of the improvement of the Missouri River above Sioux City, Iowa, where he remained until 1891, when he was given charge of improvement work in Yellowstone National Park. He carried on this work until 1893, when he was assigned to duty on the Louisville and Portland Canal.

In the fall of 1894, Capt. Chittenden was made Executive Officer of the Board of Engineers and had charge of a canal survey between Lake Erie and the Ohio River. This assignment was terminated in 1896, when he was made Secretary of the Missouri River Commission, in personal charge of the improvement of the Osage and Gasconade Rivers in Missouri and the surveys on the Missouri River, as well as surveys for reservoir sites in Wyoming and Colorado.

When war with Spain was declared in 1898, Capt. Chittenden was appointed Chief Engineer of the Fourth Army Corps, with the rank of Lieutenant-Colonel of Volunteers. In this position he had charge of the purchase of engineering supplies, the laying out of camp grounds, supplying them with water, etc. While encamped at Huntsville, Ala., he took charge of the municipal improvement of the Huntsville Spring and the erection of the new water-works plant.

At the close of the Spanish-American War, Capt. Chittenden was made Secretary and Disbursing Officer of the Missouri River Commission, in charge of the improvement of the Missouri River from Sioux City to its source, of the Osage and Gasconade Rivers, and of Yellowstone Park, with headquarters at Sioux City, Iowa. It was while on this work that he laid out the wonderful roads of Yellowstone Park and wrote a very interesting book† describing the Park itself.

* Memoir prepared by the Secretary from information on file at the Headquarters of the Society.

† "Yellowstone National Park, Historical and Descriptive."

In 1904 he was promoted to the rank of Major and transferred to Seattle, Wash., in charge of the United States Engineer Office and work in that district. His most important work while in this position was the planning of the Lake Washington Canal, a project then being agitated by the citizens of Seattle. A plan had already been made for this Canal, which included two locks, at an estimated cost of about \$7 000 000, which was prohibitive. After some study of the question, Maj. Chittenden submitted a plan with only one lock and at a cost which enabled the project to be carried out. The Lake Washington Canal is a ship canal about 7 miles long, connecting Puget Sound with Lake Washington and outside of the Panama Canal is said to be one of the most important canals built by the Government, the lock being second in size only to those at Panama. He also served as a member of the Federal Commission on the Yosemite National Park in 1904, and on a commission of engineers appointed to investigate the question of the Sacramento Flood Control.

In 1908, Maj. Chittenden was promoted to the rank of Lieutenant-Colonel and in 1910 he was made a Brigadier-General, but was retired shortly afterward owing to partial paralysis, brought on by shock sustained by a long test ride which, with all Army Officers, he was compelled to take at that time.

After his retirement, Gen. Chittenden devoted himself to consulting engineering work and investigations. On September 5th, 1911, he was elected a member of the Port Commission and was made the first President of the Port of Seattle, which position he held until October 15th, 1915. This Commission had been organized under the laws of the State of Washington, to take up the question of a harbor for Seattle; Gen. Chittenden devoted himself to this work and by his commanding personality gained the respect of those associated with him to such an extent that the project was carried out and the Port of Seattle, with its dock and terminal facilities, was built at an expenditure of about \$6 500 000. The traffic through this port to Siberia has been so enormous during the last two or three years that the docks have been found to be too small, thus vindicating Gen. Chittenden's judgment.

His project for a 30-mile tunnel through the Cascade Mountains,* although this tunnel was never constructed, was widely discussed, and his articles thereon were translated and published in both Spanish and French technical periodicals. He was also greatly interested in flood-control work, to the study and literature of which he had devoted much time. In this connection he was retained in a consulting capacity by the Spring Valley Water Company of San Francisco, Cal., in 1912, and

* *Engineering News*, November 16th, 1916, p. 928.

also by The Miami Conservancy District, to report on that project. He visited Dayton, Ohio, on several occasions during 1914-15, and being unable to walk, he was taken to all sites of the work by automobile, most of his research work being done in his rooms; in this manner he made a detailed study of the entire project. Notwithstanding his disability, Gen. Chittenden had an enormous capacity for hard work, and his mind was exceedingly active. It is said of him that when engaged on a project, he could keep many men busy supplying him with information and data in connection with it and that he himself could work steadily from 8 to 12 hours a day for 7 days in the week.

He devoted much of his time to writing, and besides being a frequent contributor to the publications of this Society, he published many articles, especially on waterways, flood control, and water supply, in various technical periodicals. Among his papers before this Society were the following: "Reservoir System of the Great Lakes of the St. Lawrence Basin";* "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers";† and "Ports of the Pacific".‡ His last paper, "Detention Reservoirs with Spillway Outlets as an Agency in Flood Control",§ was presented before the Society shortly after his death, at the meeting of October 17th, 1917. He had also discussed various papers presented by other engineers.

Most of Gen. Chittenden's life was spent in the routine engineering work of the Army Corps, in charge of public works in various parts of the United States. His fine personal qualities and his fair treatment of the men under him and the public whom he served, had won for him the respect and admiration of every one who knew him, particularly in the Northwest, where he had spent many of his latter years. An invalid for many years and confined to a wheel chair during the last year of his life, he had developed the human side of his character until his personality affected for good all who came under his influence and made him an honor to the Army and to his Profession.

In a tribute to him, and in appreciation of his qualities as a man and as an engineer, before the Society at its meeting of October 17th, 1917, Charles Evan Fowler, M. Am. Soc. C. E., said, among other things:

"I found him to be the most fair and one of the finest men to work under whom I have ever known * * *. That whether it pleases engineers or not to realize it, their work is for posterity. Gen. Chit-

* *Transactions*, Am. Soc. C. E., Vol. XL, p. 355.

† *Transactions*, Am. Soc. C. E., Vol. LXII, p. 245.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 155.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 1473.

tenden has not lived to see the flood control work carried out; he has not lived to see his tunnel built; but posterity will, without doubt, thank him for what he has done."

He is survived by his widow, who was Miss Nettie M. Parker, of Arcade, N. Y., whom he married on December 30th, 1884, and by two sons, Lieut. Hiram M. Chittenden, Jr., 14th Field Artillery, U. S. A., and Theodore Chittenden, and a daughter, Mrs. J. B. Cress.

Gen. Chittenden was elected a Member of the American Society of Civil Engineers on February 7th, 1900.

OTIS FRANCIS CLAPP, M. Am. Soc. C. E.*

Died March 3d, 1917.

Otis Francis Clapp was born in Boston, Mass., on September 26th, 1843, and was educated in the public schools and Hunt's Academy at North Bridgewater, now Brockton, Mass.

He began his engineering training in the office of Messrs. Shedd and Edson, in Boston. In 1867 he was sent by that firm to Providence, R. I., to make the preliminary surveys for the first municipal water-works system. The project, which was afterward adopted, provided for a water supply from the Pawtucket River.

In 1872, Mr. Clapp was given charge of the preparation of the plans for a sewerage system for the City of Providence, and, later, the greater part of the city's sewers and sewage disposal works was constructed under his direction.

In May, 1897, he was appointed City Engineer to succeed the late J. Herbert Shedd, M. Am. Soc. C. E. Mr. Clapp held this office for 18 years, until 1916, when he was made First Assistant in the City Engineer's Department on special work in connection with the improvement of the Moshassuck River channel. He was engaged on this work for nearly a year, but was obliged to resign on account of his failing health. Altogether, Mr. Clapp had been connected with the engineering work of the City of Providence, particularly that part of it relating to the water-works and sewerage departments, for more than 50 years.

He was a man of pleasing personality, modest and unassuming in manner, and of a genial temperament which attracted to him many friends.

In November, 1869, he was married to Anna Isabella Sweetland, whom he survived about two years.

Mr. Clapp was a member of the Boston Society of Civil Engineers. He was elected a Member of the American Society of Civil Engineers on March 2d, 1898, and at the time of his death was serving as a member of the Board of Direction, to which office he had been elected on January 19th, 1916.

* Memoir prepared by the Secretary from information furnished by William D. Bullock, M. Am. Soc. C. E.

ELMER ELLSWORTH COLBY, M. Am. Soc. C. E.*

DIED SEPTEMBER 27TH, 1917.

By the death of Elmer Ellsworth Colby, the Engineering Profession loses a man of merit and distinction and his associates a friend in every sense of the word. He was born in Oconomowoc, Wis., on July 2d, 1861, and was educated in the public schools of Wisconsin, Iowa, and Illinois. He was graduated from a Chicago High School in 1879.

Mr. Colby's professional life commenced in his nineteenth year when he began service as a Rodman on the construction of a Chicago, Milwaukee, and St. Paul Line in Carrol County, Iowa. His advancement was rapid, for, in March, 1883, he was made Division Engineer of the Kansas City, Springfield, and Memphis Railway, in charge of construction on the Memphis Division, Terminal Yards, and the Mississippi River Transfer.

After the termination of this work in 1884, and until the latter part of 1888, Mr. Colby was engaged in various construction and locating capacities in Iowa, Missouri, and Kansas, changing occupation frequently, as was the custom at that time, but always advancing, and broadening his experience. In November, 1888, however, he began work in that branch of his Profession which was destined to be his life's work, and for which he was best known among his later associates.

At that time he opened an office for the private practice of bridge designing and construction in Springfield, Mo., acting at the same time as County Engineer for Greene County, Missouri. As was to be expected, the first years of such an enterprise were a bitter struggle, and it was necessary for him to do land work, sub-division surveys, and any work of that nature that would come to a young engineer starting his own business at such a time, but he steadily continued toward his goal, and built, during this period, several bridges across the larger streams in the mountainous portion of Missouri, and, also, after entering a competition, the War Department awarded him the contract for designing and supervising the construction of a road from Springfield to the National Cemetery. He also built the unique Massey Building in Springfield, a four-story structure carried on a stone arch spanning Wilson Creek, and designed and built the concentrating mills and shafts for several lead and zinc plants, as well as doing prospecting and underground work for the companies.

When, in 1900, the United States Department of the Interior started the work of platting segregated town sites in the Chickasaw Nation, in Indian Territory, Mr. Colby was placed in charge of the work, and moved to Chickasha, where he resided until his death. After the town

* Memoir prepared by Max L. Cunningham, M. Am. Soc. C. E.

site work was completed, he began again his private practice, and, during 1903, was Chief Engineer of the Colorado, Oklahoma, and Texas Railway, a Santa Fé property connecting the main north and south lines with the main line to the Southwest through the State of Oklahoma.

The private practice opened by Mr. Colby in Chickasha included the supervision by him, as City Engineer, of all public work in that city, covering the expenditure of approximately $1 500 000. He also acted as Consulting Engineer for other cities in Oklahoma, on various municipal improvements, notably sanitary and paving work.

At the time of his death, Mr. Colby was also County Engineer for Caddo and Grady Counties, Oklahoma. He had recently completed a most important and difficult bridge across the South Canadian River, at Canadian, Tex., and his services as an expert on construction and cost were in great demand.

He was instrumental in securing the present highway and sanitary laws of Oklahoma, and was one of the founders of the Oklahoma Society of Engineers, having served as its President in 1910. He belonged to the B. P. O. E., and was also an active member of the American Society of Municipal Improvements and of the American Association for the Advancement of Science.

Professionally, Mr. Colby was a marked man in his community, and his interest in civic affairs made him a leader, both in Chickasha and in the State. His death was hastened by application, as a citizen, to a matter that was absolutely valueless to him as an engineer or to his business. The incident was the culmination of a series of public services extending over many years and appreciated more after his death than before it.

He leaves a widow, two sons, both of whom are Civil Engineers—the younger of whom is taking up his father's work so far as he can—six daughters, and a host of friends, all of whom feel his loss as a personal one. He died at work, having been stricken by heart failure while at the wheel of his car on his way to inspect some work in his charge. His last words were an inquiry as to whether he was "on the right road."

It is difficult for the writer to express his feelings at the loss of a friend such as Mr. Colby had been for years, but it seems to him that any engineer who can live as full and useful a life as Mr. Colby had, who can become so valuable a man to the State, and can be so universally mourned, has accomplished his life's work in an enviable manner and could well be proud to die in harness with his hand to the wheel as he did.

Mr. Colby was elected a Member of the American Society of Civil Engineers on January 7th, 1913.

CHARLES LEE CRANDALL, M. Am. Soc. C. E.*

DIED AUGUST 25TH, 1917.

Charles Lee Crandall was born on a farm near Bridgewater, N. Y., on July 20th, 1850. He prepared for college at Whitestown Seminary, Whitestown, N. Y. At the opening of Cornell University his father purchased a farm at Ithaca, N. Y., the seat of that institution, and moved there to educate his children. This was ever afterward Mr. Crandall's home. He entered Cornell as a freshman in 1868, and was graduated in Civil Engineering with the class of 1872, the first four-year class.

After two years of professional practice in geodetic and railroad work, on returning to Cornell for advanced study, he was induced by Professor Fuertes, then head of the department of Civil Engineering, to act as Assistant in teaching, while pursuing his advanced work; and at that time these two men were the entire instructing force in that department.

On August 20th, 1878, Professor Crandall married Miss Myra G. Robbins, of Bridgewater, N. Y., who survives him. Their home life was ever ideal, and their mutual devotion complete. To the "boys" of the College of Civil Engineering their home was a refuge to which they could turn, and return, for advice, for inspiration, and for substantial aid.

Professor Crandall's first teaching was in descriptive geometry. One of the writers was of his first class, and his recollection remains yet most keen of that earnest, honest, modest face. He was more the elder brother than the professor. In our boy parlance we pronounced him "all right", that first term, and have loved him ever since. As a teacher of field work there has never been his equal. His students have proved this in their life work, especially when first leaving his instruction. In those early days each professor was obliged to teach many subjects. Professor Crandall's principal work was in geodesy and railroads, but he taught all field work throughout the student's training.

In 1875 he was made Assistant Professor. In 1876 he took his degree of Master of Civil Engineering, and in 1891 was appointed Associate Professor. He was advanced to the position of full Professor in 1895, and from 1902 to 1906 was Professor in Charge of the College of Civil Engineering. In 1915, having reached the age of retirement, he was made Professor Emeritus.

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* Memoir prepared by the following Committee: Willard Beahan and James H. Edwards, Members, Am. Soc. C. E., and Irving P. Church, Assoc. Am. Soc. C. E.

The following is from an obituary notice of Professor Crandall:*

"Professor Crandall retained a warm, almost parental, interest in the graduates of the College of Civil Engineering. Through a correspondence bureau, which he conducted for many years, he obtained professional employment for many of these men. He kept the alumni records of the college, which were a model of accuracy and completeness. The Cornell Society of Civil Engineers gave a dinner in his honor in New York City in January, 1916, and established the Charles Lee Crandall Prize in the College of Civil Engineering, besides having his portrait painted, which was presented to the University in June, 1916."

Professor Crandall's authorship of technical books embraces a number of works, on a variety of subjects, prepared for the use of his students. His well-known "Text-Book on Geodesy and Least Squares" was published in 1907, and "Tables for the Computation of Railway and Other Earthwork" first appeared in 1886, reaching a fifth edition in 1916. He also prepared "Notes on Descriptive Geometry" and "Notes on Shades, Shadows, and Perspective". The "Transition Curve" was first published in 1893, a second edition in 1899, and was incorporated as a special chapter in the "Field Book for Railroad Surveying", of which Professors Crandall and Barnes were joint authors. "Railroad Construction", also by these two men jointly, was published in 1913. In addition to these books, Professor Crandall contributed many articles to engineering periodicals.

The following is taken from an article† by F. A. Barnes, Assoc. M. Am. Soc. C. E., Professor Crandall's colleague in the College of Civil Engineering at Cornell:

"Professor Crandall was a member of the honorary scientific society, Sigma Xi, since 1887, and of the honorary society of Tau Beta Pi since the formation of a chapter at Cornell a few years ago. He was also a member of the Civil Engineering Society, Semaphore, and of the Zodiac Fraternity."

"As a member of many engineering and scientific societies and other organizations, Professor Crandall took an active interest in their work. * * * He was a member of the Society for the Promotion of Engineering Education from its foundation in 1893, served as its President in 1906, and had since been on its Council as a Past-President, *ex officio*. At the time of his death he was also one of its representatives on the Joint Committee on Engineering Education."

"But perhaps his best and most extensive committee work was on the Committee on Iron and Steel Structures of the American Railway Engineering Association, on which he had served since he joined the Association in 1901, a year or two after its foundation. Since 1907 he, with Professor F. E. Turneaure, Dean of the College of Engineering of the University of Wisconsin, and others, had been

* *Cornell Alumni News*, September, 1917, p. 479.

† *Cornell Civil Engineer*, October, 1917, pp. 1–8.

investigating the question of impact on railroad bridges. Several summer trips had been made for the purpose of securing experimental data, and plans had already been made for another in September, 1917. The data already obtained had been worked up, and a new impact formula based on the results had been devised."

"Professor Crandall was ever mindful of his civic responsibilities, and, because of his intimate knowledge of the city, obtained through his long term of service as City Engineer, was often called upon by the authorities for advice. He had served on the Sewer Commission, was a member of the committee appointed to revise the charter a few years ago, and was serving a term as Commissioner of Public Works at the time of his death."

The following paragraph forms a portion of the resolutions adopted by the Cornell University Faculty, in June, 1915, on the occasion of Professor Crandall's retirement from teaching:

"But what was probably Professor Crandall's greatest usefulness was the result of his high unselfish character. His whole life had been given to the devoted service of his associates and of his students. Graduates of the College of Civil Engineering have no memories that do not include a feeling of affection and thankfulness to Professor Crandall. Of kindly disposition and practical sound sense, sympathetic in his intercourse with students, quiet and modest in manner, but with strong convictions as to truth and justice in any matter brought before him, and always ready to sacrifice personal interests in following the dictates of duty, Professor Crandall had won the warm esteem of all who had come within the circle of his influence during his two score years of service at Cornell."

Professor Crandall was elected a Junior of the American Society of Civil Engineers on June 7th, 1876, and a Member on October 5th, 1892.

JAMES AUBREY DAVENPORT, M. Am. Soc. C. E.*

Died March 15th, 1918.

James Aubrey Davenport was born at Athens, N. Y., on February 27th, 1858. He received his early education from a private tutor. After some engineering training, he entered the employ of the Shenandoah Valley Railroad Company, with headquarters at Charlestown, W. Va., serving as Chainman, Rodman, and Instrumentman, in 1878, 1879, and 1880, when he was made Assistant Engineer. He remained with the Company in that capacity until the completion of the line to Roanoke, Va., in 1881.

He was then appointed Assistant Engineer on the Norfolk and Western Railroad and, as Resident Engineer, had charge of important work in connection with the grading of grounds and construction of buildings for the Roanoke Machine Works, a subsidiary of the Norfolk and Western Railroad Company, now known as the Roanoke Shops of that Company. From 1885 to 1887, he was employed as Assistant Engineer on surveys for the Railway Company.

From 1887 to 1890, Mr. Davenport served as Roadmaster and Division Engineer, on maintenance of way, on the Richmond and Danville Railroad (now the Southern Railway). In 1890, he was transferred from the Western North Carolina Division to the Georgia Pacific Division, of the same road, where he remained as Division Engineer on maintenance of way until 1895, when he was appointed Assistant Engineer on the construction of the Central of Georgia Railway.

In 1900, Mr. Davenport returned to the Norfolk and Western Railroad as Assistant Engineer, and was put in charge of work, under W. W. Coe, M. Am. Soc. C. E., then Chief Engineer, incident to railway surveys and double-tracking. On January 1st, 1903, Charles S. Churchill, M. Am. Soc. C. E., succeeded Mr. Coe as Chief Engineer, and Mr. Davenport was continuously employed, under the Chief Engineer of the Norfolk and Western Railway, in charge of heavy tunnel work and double-track and branch line construction of importance, until September, 1916, when he was granted an indefinite leave of absence on account of ill health. He died at his home in Roanoke, Va., on March 15th, 1918, and was buried at Charlestown, W. Va. He is survived by his wife and four sons.

In the record of his services to the Norfolk and Western Railroad, it is stated that Mr. Davenport was "a most faithful and competent employee", and that he performed his duties as Assistant Engineer with ability and fidelity to his employers.

Mr. Davenport was elected a Member of the American Society of Civil Engineers on April 5th, 1905.

* Memoir prepared by the Secretary from information on file at the Headquarters of the Society.

HARRY MADERA GOULD, M. Am. Soc. C. E.*

DIED SEPTEMBER 30TH, 1917.

Harry Madera Gould, the son of John E. and Cora D. Gould, was born at Worthville, Ky., on April 12th, 1872. He received his grammar and high school education at Worthville and Louisville, Ky., and was graduated from Rensselaer Polytechnic Institute, in the class of 1895, with the degree of Civil Engineer.

He entered railroad work with the "Big Four" Railroad at Louisville, Ky., but moved to Colorado, where he was engaged for a while in mining work. In 1897, he was employed by the Louisville and Nashville Railroad Company as Assistant Engineer. From 1898 to 1905, he was associated with his father in the erection of steel bridges, principally railroad structures, and, from 1906 to 1908, he was Secretary and Treasurer of the Gould Construction Company, doing the same class of work. In 1907 and 1908 the Gould Construction Company, in conjunction with the Foster and Creighton Company, erected the two steel and concrete highway bridges over the Cumberland River at Nashville, Tenn., and Mr. Gould was General Manager of the work. From 1909 to 1914 he was Vice-President and General Manager of the Foster-Creighton-Gould Company, during which time the company built the Kentucky and Indiana Railroad bridge foundation in the Ohio River, at Louisville, erected the Fort Smith and Van Buren Highway Bridge superstructure across the Arkansas River, built the Louisville and Nashville Railroad Bridge complete across the Cumberland River, at Nashville, and a number of railroad bridges and other structures. From 1915 to 1917 he was President of Gould Contracting Company, doing steel erection and foundation work, having just completed a power dam across Caney Fork River, Tennessee; he had also been engaged on the construction of the Hydes Ferry Bridge over the Cumberland River, at Nashville, Tenn., now nearing completion.

Mr. Gould had been in bad health for several months and died at Nashville, Tenn., on September 30th, 1917. His wife and one child survive him.

He had been a member of the Engineering Association of the South (now the Engineering Association of Nashville) since May, 1903, and had served as a Director and as President. He was a man of high business principles and ability, whom not only his friends, but the community, will miss.

Mr. Gould was elected a Member of the American Society of Civil Engineers on November 30th, 1909.

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* Memoir prepared by C. B. Wilson, M. Am. Soc. C. E.

HENRY AUGUSTUS HERRICK, M. Am. Soc. C. E.*

Died December 14th, 1917.

Henry Augustus Herrick was born on November 26th, 1861, in Brooklyn, N. Y. His parents, Henry Walker Herrick and Clarissa Harlow Parkinson, moved to Manchester, N. H., when he was four years old. He was educated in the public schools in Manchester, and was graduated from the High School at the age of nineteen.

His first work after graduation was with the Amoskeag Manufacturing Company, at Manchester, in connection with the design, construction, and equipment of cotton mills. He was later, in succession, engaged in similar work at the Methuen Mills, Methuen, Mass., at Willimantic, Conn., with F. P. Sheldon, Mill Engineer, Providence, R. I., and at the Schuylerville Mills.

In 1890 he went to Spokane, Wash., where he supervised the construction of the dam and power station of the Washington Water Power Company, which is still in operation.

On January 14th, 1891, Mr. Herrick was married and went to Great Falls, Mont., where he designed and supervised the construction of the power plant for the Boston and Montana Smelter.

In the latter part of 1895 he became associated with Dean and Main, Engineers, and was engaged in the design and supervision of construction of many textile and other industrial plants, and, with the dissolution of this firm in 1907, he continued his association with the writer.

The most interesting, valuable, and effective work was done by Mr. Herrick in the last 10 years of his life, during which he had charge of the design and construction of some of the largest and best hydro-electric developments in the country, namely, the Rainbow Falls development and the Great Falls development, on the Missouri River near Great Falls, Mont., the Thompson Falls development, on Clarks Fork of the Columbia River, at Thompson Falls, Mont., and he was just nearing the completion of his work on the Holter development, near Helena, Mont., when he was stricken by heart failure and died immediately. All these developments formed a part of the system of the Montana Power Company, and aggregated about 250 000 h. p.

During the time that the foregoing work was in progress, and between the ending of one and the beginning of another undertaking, he made many examinations and reports on other projects, and acted as Consulting Engineer on several.

* Memoir prepared by Charles T. Main, M. Am. Soc. C. E.

Mr. Herrick had developed a systematic analysis of dams, based on his many years of close study and experience in designing such structures. All the many fundamentals, such as geological formation, seepage, drainage, upward pressure, shock and dissipation of energy due to overflow, flood conditions, etc., received proper consideration in his analysis.

His early training in design and construction enabled him to design the buildings for power stations in a most acceptable manner, and he mastered the electrical problems with great ease.

He was a great student and a fine mathematician. He was one of the best types of self-educated engineers. His character was of the highest, and his dealings with men above and below him were always characterized with fine courtesy and uprightness.

His principal recreation was trap and target shooting, hunting, and fishing, and he took as much pleasure in anticipation of outings as in the rare actual experiences which his busy life allowed to him.

He was a member of the Boston Society of Civil Engineers, the Engineers Club, of Boston, the Fresno Club, of Fresno, Cal., the Silver Bow Club, of Butte, Mont., and other societies.

Mr. Herrick was elected a Member of the American Society of Civil Engineers on May 7th, 1890.

OLIVER ZELL HOWARD, M. Am. Soc. C. E.*

Died December 20th, 1917.

Oliver Zell Howard was born in Raleigh, N. C., on December 6th, 1876. He was graduated from Lehigh University in the Class of 1897, with the degree of M. E.

Immediately after his graduation in June, 1897, Mr. Howard was appointed Draftsman in the Engineering Department of the Newport News (Va.) Shipbuilding and Dry Dock Company, where he remained until January, 1900, at which time he became Draftsman for the Baltimore (Md.) Copper Smelting and Refining Company, in charge of rolling mill construction and repair.

From November, 1900, to August, 1901, Mr. Howard was engaged as Draftsman in the Marine Department of the Maryland Steel Company, at Sparrows Point, Md., on the design of traveling towers for ship construction.

From August, 1901, to April, 1902, he was connected with the Bureau of Construction and Repair of the United States Navy Department, as Draftsman in charge of the design of auxiliary machinery, but resigned that position to become Instructor in the Department of Marine Engineering and Naval Construction, at the United States Naval Academy, at Annapolis, Md. He also had charge of the general layout, the preparation of the specifications, and the installation of the mechanical and electrical equipment of the new Marine Engineering Building at the Academy, and assisted the officer in charge of the Department in preparing textbooks on marine engines and machinery and on the mechanical processes pertaining thereto for use at the Academy.

From August, 1906, to February, 1912, Mr. Howard was engaged as Mechanical Engineer and Civilian Assistant to the Head of the United States Naval Engineering Experiment Station at Annapolis, Md., and had charge of the design, inspection, and installation of the mechanical and electrical equipment during the construction of the station buildings, having charge, after their completion, of all the electrical and mechanical tests conducted at the Station.

In February, 1912, he was appointed Assistant Chief Engineer of The Griscom-Russell Company, of New York City, which position he retained until March, 1914. During this time he designed and built garbage disposal plants for Halifax, N. S., and Berkeley, Cal., and a refuse disposal plant for Pittsburgh, Pa., etc.

* Memoir prepared by the Secretary from information on file at the Headquarters of the Society.

In March, 1914, he established himself in private practice, as a Consulting Engineer with an office in New York City. Among other work he designed and constructed a chemical plant for the Diamond Match Company, at Wilmington, Cal., as well as a chemical plant for the manufacture of potash for the Salt Lake Chemical Company, at Burmester, Utah.

Early in 1917, Mr. Howard was employed as Supervising Engineer by the Diamond Match Company, with headquarters in New York City, which position he retained until his death on December 20th, 1917.

He was a member of the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, and the Society of Naval Architects and Marine Engineers.

Mr. Howard was elected an Associate Member of the American Society of Civil Engineers on February 28th, 1911, and a Member on May 15th, 1917.

CLARENCE BOOTH LAMONT, M. Am. Soc. C. E.*

DIED MARCH 21ST, 1918.

Clarence Booth Lamont was born in Van Etten, N. Y., on February 23d, 1877, his parents having been Clarence Lamont and Iva A. Booth Lamont. After receiving a common school education, he entered Cornell University, from which he was graduated in 1900, with the degree of Mechanical Engineer.

Before going to college Mr. Lamont had evinced a strong interest in machinery and mechanical work. From June, 1894, to September, 1896, he had been engaged with the Singer Manufacturing Company on general office work and as Assistant on construction, and, during his college vacations, he found employment at Cramp's Shipyards, at Philadelphia, Pa., and in other shops. All this practical training was of great benefit to him in his theoretical work in college, and provided him with splendid equipment for his future career.

After his graduation Mr. Lamont began work at the great shipbuilding and engineering plant of the Union Iron Works, at San Francisco, Cal., which, even at that time (1900), was the best organized and equipped plant of the kind on the Pacific Coast. His work in designing for this Company continued from July, 1900, to February, 1901, and the experience gained in this capacity enabled him to make rapid progress in subsequent work. He then entered the service of the Government as Hull Draftsman for the United States Navy Department, where he was employed until May, 1901.

The expansion of the plant of the Moran Brothers Company, at Seattle, Wash., where the battleship *Nebraska* was built, gave Mr. Lamont a great opportunity. He entered the employ of that Company in June, 1901, as Assistant Engineer and Designer in charge of several other designers, and his intelligent work was no small factor in the development of the present great works.

The Company operating the White Pass and Yukon Railway, running from Skagway, Alaska, to White Horse, on the Yukon River, owned a large fleet of stern-wheel steamboats which ran, during the summer, from the latter point to Dawson, Yukon Territory, and, from February to November, 1902, Mr. Lamont was engaged as Assistant Superintending Engineer of the River Division of the Railway Company on new construction work and the betterment of the boats in operation. Thus, it was a logical step in advance when, in October, 1902, he accepted the position of Port Engineer for the great fleet of the Pacific Coast Steamship Company, which has its headquarters at Seattle. He remained with this Company until March, 1904, having

* Memoir prepared by Charles Evan Fowler, M. Am. Soc. C. E.

had the supervision of from 10 to 14 ocean-going ships, and showed much ability in obtaining the greatest efficiency in operation. Among the many devices for the transmission of power on vessels, which he worked out during this period, was an invention for absorbing the propeller shock on ships driven by screw propellers, which permitted an increase in speed without damage to the framework of the vessel.

Having to do with commercial affairs in his steamship work, he became attracted to pure business, and, during a portion of 1904, acted as Manager of the Seattle Branch of the Plant Rubber Supply Company. This experience resulted in the formation of the Pacific Engineering Company, of which he was President, having as his associates, G. Boschke, Assoc. M. Am. Soc. C. E., and Mr. Frank W. Hibbs, who had previously served as a Naval Constructor for the United States Navy Department. The firm was engaged in the supply business and also in general contracting, in which latter work Mr. Lamont showed the greatest initiative and adaptability. Among the important contracts handled by the firm were the large ocean docks or wharves at Portland, Ore., for the Spokane, Portland, and Seattle Railway. This railway is commonly known as the North Bank Road, and is a low-grade line running from the East down the Columbia River into Portland, requiring the docks for handling its ocean commerce. This contract was carried out in a very efficient and skillful manner, and placed Mr. Lamont in the front rank of constructors in the Northwest.

In the meantime the Moran Shipbuilding Plant had been financed by Eastern capital, and had become the present great Seattle Construction and Dry Dock Company. Mr. Lamont was asked to return to its management, which he did in November, 1908, as Assistant to the President. The Company was handling about \$2 000 000 worth of shipbuilding and ship repair work annually, and he assisted in a large way in building up the efficiency of the works to its present standing as one of the most dependable plants engaged in war-time shipbuilding. Mr. Lamont made many trips to Washington, D. C., in the interest of the Corporation and was successful in securing from the Government many large contracts for warships.

During this period many important engagements came to him as Consulting Engineer, including that of Trustees' Engineer for the White Pass and Yukon Route, and nothing speaks more in praise of his ability and energy than his re-employment by two great corporations for which he had previously worked. He resigned his position with the Seattle Construction and Dry Dock Company in 1915 and organized the firm of C. B. Lamont, Incorporated, for carrying out large projects and acting as consulting engineers. He was one of the organizers of the Skinner and Eddy Shipbuilding Corporation, of Seattle, one of the first firms to enter the business of building steel

cargo vessels for the Allies, and, to a great extent, the wonderful records made by this firm were due to his ability and foresight.

He was engaged in such work in an energetic and forceful way up to the time of his death, and, although business occupied much of his time, he never failed to take great interest in public affairs and in social life. He was one of the organizers of the Seattle Hunt Club, and was a great horseback rider and a most enthusiastic sportsman and hunter. He was also a member of the Seattle Golf and Country Club, the Rainier Club, the Seattle Athletic Club, the Seattle Chamber of Commerce, the Union Club of Tacoma, Wash., and the Chevy Chase and Metropolitan Clubs of Washington, D. C.

Mr. Lamont was married in 1908 to Miss Maud Hahn, who survives him. Mrs. Lamont was a great help to her husband and shared in his leadership of many social affairs.

The best monument he has left behind is the unswerving regard of his business associates and the undying devotion of his friends. The Engineering Profession has suffered a distinct loss by his untimely death which came suddenly from pneumonia, on March 21st, 1918, in the busiest period of his life, when he was just past his forty-first birthday, and with his life a success:

"For this is Art's true indication,
When skill is minister to thought,
When types which are the mind's creation,
The hand to perfect form hath wrought."

Mr. Lamont was elected an Associate Member of the American Society of Civil Engineers on May 4th, 1909, and a Member on June 24th, 1914.

FRANKLIN BUCHANAN LOCKE, M. Am. Soc. C. E.*

Died May 11th, 1917.

Franklin Buchanan Locke was born at Hampton, N. H., on February 23d, 1857. He came of New England ancestry and was a lineal descendant of Captain John Locke, described as a man of prominent position among the early settlers of New Hampshire, who was killed by the Indians in 1696.

Franklin Buchanan Locke received his engineering education at the Massachusetts Institute of Technology, where he was a member of the class of 1877. He discontinued his course, after three years, however, to engage in the work of construction of the Hoosac Tunnel. His elder brother, the late Augustus W. Locke, M. Am. Soc. C. E., also an eminent engineer, was engaged on the same work, and, after the completion of the tunnel, became Superintendent of the Troy and Greenfield Railroad. Franklin Buchanan Locke was Assistant Engineer on the tunnel until 1881, at which time he formed a partnership with his brother, and the firm became known as perhaps the leading engineering firm of Western Massachusetts.

Mr. Locke's work was of great variety, covering the fields of railroad, municipal, and even mining, engineering. One of his earliest engagements was as Engineer for the extension of the water-works and drainage system of North Adams, Mass. He also had charge of the double-tracking of the State road east of the Hoosac Tunnel, and of the masonry.

In 1881-82 he was Principal Assistant Engineer on the Buffalo, Rochester and Pittsburg Railway, on that portion of the line between Salamanca, N. Y., and Ridgway, Pa. He did some valuable and interesting location work on the Mountain Division, south of Bradford, Pa., through a country parallel to the line of the Erie Railroad over the Kinzua Viaduct. He resigned this position, owing to poor health, and spent some time traveling in Europe, visiting most of the great Alpine tunnels and inspecting European railways. Returning to the United States, he became Engineer of Maintenance of Way of the Troy and Greenfield Railroad, which embraces the Hoosac Tunnel. He was also Chief Engineer of the Hoosac Valley Electric Railroad, which runs through Adams, North Adams, and Williamstown, Mass., and of a branch of the Fitchburg Railroad, which he constructed. He also investigated and reported on numerous railroad and mining properties in the West, and, for a time, was Engineer for the Michigan Hydraulic Mining Company in Idaho, as well as for some mines in Utah. With

* Memoir prepared by George F. Swain, Past-President, Am. Soc. C. E.

his brother, Mr. Locke became greatly interested in the problem of abolishing railroad grade crossings in Massachusetts, and was connected with projects for several works of this kind.

Finally settling in North Adams, Mass., he held the office of City Engineer for a number of terms, and also the office of Commissioner of Public Works; indeed, for many years, he was depended on to take the lead in the engineering work of the city. While in these positions, Mr. Locke had to do with the construction of water-works, sewers, roads, and bridges. He built many roads, and was an authority on their construction. He was, probably more than any one else, responsible for the successful recent extension of the North Adams Water-Works, which he planned and began.

One of Mr. Locke's most recent activities was in connection with the location and construction of the so-called "Mohawk Trail", a State road across the Berkshire Hills, between North Adams on the west and the Deerfield Valley on the east. Indeed, he was known in the vicinity of North Adams as "the Father of the Mohawk Trail". Although he started the movement for this road and surveyed a line, his route was not adopted by the State officials, although many considered it the best line; but, at any rate, Mr. Locke first conceived the idea of this trail.

He had traveled much in the United States and abroad, had assimilated his experience, and had gathered a great deal of knowledge. He had read widely and intelligently. Some people travel and read, but do not learn; Mr. Locke was not of this type. His breadth of interest was greater than that of most engineers. He had an artistic nature, and had even given some attention to art. He was modest, retiring, and, if he had any fault, it was that he did not push himself to the extent that his ability warranted, for he was really a man of fine perception, strong common sense, and exceedingly capable as an engineer. With greater stimulus, he might have made himself better known, and might have achieved a greater measure of what is usually, though probably mistakenly, termed success; but he preferred the quiet retirement of his home and the society of his friends and of his books, and, probably, after all, he derived in that way a greater measure of real satisfaction than falls to the lot of those who are more prominent or pushing in the battle of life. Mr. Locke was sociable, upright, hopeful, courageous, conscientious, and true. He had a high conception of the calling of the engineer, and no one could ever say that he had failed in his duties toward his fellow-men. He made many friends and lost few, and was beloved by all who knew him. His judgment was relied on by all who had had opportunity to test it, and his death has left a vacancy among his friends and in the engineering circles in which he moved, which will be difficult to fill. Even after his health became impaired, his advice was sought and relied on. In his home town, where his friends were legion, he will be sadly missed, but as long as a good water supply

and the beauties of Nature are appreciated, his name will be held in affectionate remembrance.

Mr. Locke had been in failing health since 1907. He had never been extremely robust; nevertheless he continued to serve his State to the extent that his strength would permit. In 1914, when his health broke down and he had to give up public responsibility, he resigned as Commissioner of Public Works. A trip South benefited him for a time, but afforded no permanent relief, and he died, quite suddenly, in a hospital in Boston, Mass., when his friends thought he was out of danger.

He was a member of the American Institute of Mining Engineers, the Boston Society of Civil Engineers, the North Adams Club, and the Berkshire Club.

Mr. Locke was elected a Member of the American Society of Civil Engineers on March 1st, 1893.

JESSE LOWE, M. Am. Soc. C. E.*

Died April 17th, 1918.

Jesse Lowe was born at Omaha, Nebr., on January 7th, 1861. He attended private and public schools there, later going to Maryland Agricultural College, a military academy; then to Williston Seminary, at Easthampton, Mass.; and, finally, to Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1885, with the degree of Civil Engineer.

After his graduation Mr. Lowe was, for a short time, Assistant to the City Engineer of Omaha, and Assistant Engineer in locating the Omaha Belt Line Railway, and in the preliminary and location surveys of the Missouri Pacific Railroad west of Omaha. In 1886, he was employed, at Lincoln, Nebr., as Resident Engineer of the Missouri Pacific Railroad.

He went next to Birmingham, Ala., as Assistant Manager of the Birmingham Bridge and Bolt Works. In 1887, he formed a partnership, at Omaha, with the late Andrew Rosewater, M. Am. Soc. C. E., and George B. Christie, M. Am. Soc. C. E., and was extensively engaged in civil engineering work in the Middle West. The following year this firm was dissolved, and that of Christie and Lowe, Civil Engineers and Contractors, was organized and continued until 1913. Mr. Lowe was always an active member of the firm, which was engaged in important engineering works in many parts of the United States, among which were the following:

The Cable Street Railways at Denver, Colo., in 1889; the cable lines of the Cleveland Street Railway Company, at Cleveland, Ohio, and the street railways of the Judson Pneumatic Railway Company, at Washington, D. C., in 1890; the Montague Street Cable Railway, at Brooklyn, N. Y., in 1891; the completion of the cable system at Denver in 1892-93; the piers for the Bellefontaine Bluffs Bridge over the Missouri River and the Harlem Creek Culvert, in St. Louis, Mo., for the Kansas and Northwestern Railroad, in 1892; the Fullerton Avenue Loop, in Chicago, in 1892, the first underground trolley line in America; and the V Street Railway, in Washington, D. C., which was the first in that city. The success of this last piece of work revolutionized street railway construction, and was the pioneer engineering work in doing away with overhead electric wires and placing them underground.

In 1894-95-96, two miles of the Chicago Drainage Canal and the Controlling Works were constructed by the firm, including the great

* Memoir prepared by George B. Christie, M. Am. Soc. C. E., and Jesse Lowe, Jr., Jun. Am. Soc. C. E.

bear-trap dam which regulates the flow of water from the Great Lakes to the Gulf. This contract was awarded on merit for the best design submitted for the regulation of the flow of water through the canal.

In 1897, the firm did railroad and bridge work for the Illinois Central and Louisville and Nashville Railways; levee construction on the Illinois River; and, in 1899, bridge foundations at South Chicago for the Baltimore and Ohio Railroad.

From 1898 to 1913, the firm was engaged in river and harbor improvement for the United States Government, and completed, in order, the jetties at Sabine Pass, Texas, at Calcasieu Pass, Louisiana, and at the mouth of the St. Marys River, Cumberland Sound, Fernandina, Fla.; locks and dams on the Warrior River, Alabama; jetties and improvement of South West Pass of the Mississippi River, Louisiana; closing of Pass á l'Outre and Cubits Gap, Mississippi River, Louisiana; and sea walls at Fort Morgan, and Fort Gaines, Mobile Bay, Alabama.

Mr. Lowe was always an ardent advocate, and a valiant champion, of the Lakes-to-the-Gulf Deep Waterway project, both as to its beneficial bearing on the vast acreage of adjoining valley lands and for the improvement of the transportation facilities from the Lake territory to the Gulf ports.

During his later years he was actively engaged in drainage and reclamation work on the Illinois River and in improving and managing extensive land holdings at Beardstown, Ill.

Mr. Lowe's sudden death, on April 17th, 1918, at Chicago, Ill., came as a great shock to his many friends. He leaves behind him, from coast to coast, a series of monuments which, for ages to come, will testify to his great ability and success as a civil engineer and builder.

He was a member of the Chi Phi Fraternity, Western Society of Engineers, Illinois Society of Engineers, Louisiana Society of Engineers, and the Structural Engineers Association of Illinois.

Mr. Lowe was elected a Member of the American Society of Civil Engineers on October 2d, 1895.

WILLIAM McKELVEY MARPLE, M. Am. Soc. C. E.*

DIED MARCH 20TH, 1918.

William McKelvey Marple, the son of the Rev. Abel Augustus and Harriet Neal (McKelvey) Marple, of Bucks County, Pa., was born in Wellsboro, Pa., on December 7th, 1852. His father was a clergyman of the Protestant Episcopal Church, and, in 1863, the family moved to Scranton, Pa., where, as Rector of St. Luke's Church until 1877, he was prominently identified with all the activities of that city and beloved by all who knew him.

William McKelvey Marple attended the public schools of Scranton, afterward studying Civil Engineering at Lehigh University.

In January, 1870, he was appointed Chainman and Rodman with the Engineering Corps of the Coal Department of the Delaware, Lackawanna and Western Railroad, under Mr. John F. Snyder. Later, he served, as Assistant Engineer, in the Railroad Department of the same road, under the late James Archbald, M. Am. Soc. C. E., Chief Engineer, on the construction of several new branches, among which was the Jessup and Winton Branch.

In July, 1879, Mr. Marple was employed as Division Engineer with the Chicago and Northwestern Railway, but resigned in January, 1880, to accept a position as Mining Engineer with the Franklin Iron Manufacturing Company of Clinton, N. Y.

From August to December, 1883, he was engaged on surveys for the Edgerton Coal Company and from February, 1884, to February, 1885, he was employed by the Delaware, Lackawanna and Western Railroad Company on surveys and office work.

In March, 1885, Mr. Marple began his service with the Scranton Gas and Water Company, first as Assistant on surveys, until December, 1886, then as Assistant Engineer on the construction of dams and reservoirs, and, finally, as Chief Engineer, which position he retained until he resigned on June 1st, 1913, to engage in private practice as a Consulting Engineer.

Under the broad and far-seeing plans of its President, the late W. W. Scranton, and by his twenty-eight years of honest, tireless, and conscientious work, and his energy, loyalty, and determination to keep pace with its engineering possibilities, Mr. Marple built and expanded this water system until, to-day, the City of Scranton possesses one of the finest of water supplies. The Water Company and its subsidiaries cover a territory which extends to Forest City, about 23 miles north of Scranton, and it was under Mr. Marple's direction, as Chief Engineer, that its largest dams were designed and constructed. Among

* Memoir prepared by Homer F. Cox, Chf. Engr. and Supt., The Scranton Gas and Water Company, Scranton, Pa.

these are the Curtis Dam (1886-87), Elmhurst Dam (1887-88), Williams Bridge Dam (1892-93), Burnt Bridge, or Lake Scranton, Dam (1898-99), and the Brownell Dam, at Carbondale, Pa. (1905-06). All are gravity dams of the best type of hydraulic masonry, equipped with the latest kinds of valves and with modern masonry gate and screen chambers.

In addition to this storage and distribution system, Mr. Marple located and built nearly 10 miles of macadam roads around the several reservoirs of the Water Company and to the summits of the water-shed, the Scrub Oak Drive reaching an elevation of 2 092 ft., thus giving to the public magnificent drives for their enjoyment.

In 1908, in order to make the supply of water to the city doubly sure, a tunnel, 7 by 9 ft. and 4 300 ft. long, was driven, under Mr. Marple's direction, in solid rock, through which a 48-in. supply pipe for conveying the water was laid. In 1909, it was found necessary to treat the water supply of Providence, a suburb of Scranton, as it came from an inhabited district of the water-shed, and a filter plant of the rapid sand gravity type, consisting of six units, each having a capacity of 1 000 000 gal., was constructed under Mr. Marple's supervision.

During his long service as Chief Engineer of the Scranton Water and Gas Company, Mr. Marple had associated with him, in a consulting capacity, such men as the late Alphonse Fteley and J. James R. Croes, Past-Presidents, Am. Soc. C. E., the late E. Sherman Gould, M. Am. Soc. C. E., and others of equal prominence.

Mr. Marple continued in private practice as a Consulting Engineer until he was compelled to retire in 1917 on account of failing health. He had been confined to his bed for several months previous to his death, but was a patient sufferer at all times and most thoughtful toward all who cared for him. He died at his home in Scranton, Pa., on March 20th, 1918, and is survived by his wife, Mrs. Mary A. Marple, and one daughter, Mrs. Eugene Cornell Kelley, of Poughkeepsie, N. Y.

He was a Charter Member of the Scranton Engineers' Club, and always gave freely of his time and talents in the furtherance of its best interests. He had been twice honored by being elected its President, and just previous to his death had been made an Honorary Member. He was also a member of the American Water Works Association, the New England Water Works Association, and the American Gas Institute.

Aside from his professional associations, he was a member of the Peter Williamson Lodge, F. and A. M., the Knights Templars, and the Irem Temple. He also took a most active part in church work as a member of St. Luke's Protestant Episcopal Church, having served as a Vestryman for 16 years.

Mr. Marple was elected a Member of the American Society of Civil Engineers on June 4th, 1890.

WILLIAM WALTER MARR, M. Am. Soc. C. E.*

Died October 3d, 1917.

William Walter Marr was born on June 9th, 1876, in Chicago, Ill., and came of an old American family. He was educated in public and private schools in Chicago and at the University of Notre Dame, at South Bend, Ind., from which institution he was graduated as a Bachelor of Science in 1895. He took post-graduate work in 1896 for which he received the degree of C. E.

In 1897 he entered the service of the City of Chicago as an Assistant Engineer and for several years was employed in various capacities in charge of bridge repairs, water supply, harbor work, and tunnel construction, under the general charge of the City Engineer.

In 1902, Mr. Marr became Division Engineer for the Board of Local Improvements of the City of Chicago, in general charge of street paving work for the West Side.

In 1908, together with the writer, he formed the present civil engineering firm of Marr, Green and Company, of Chicago.

In 1914, he was appointed by Governor Dunne, a Democrat, as Chief State Highway Engineer, and, in 1917, was re-appointed to that position by the incoming Republican Governor Lowden. Under Mr. Marr's charge the State Department of Highways was practically re-organized and placed on a thoroughly efficient basis. Many millions of dollars' worth of work was planned and constructed. At the time of his death, he was engaged on far-reaching plans providing for the expenditure of some $60 000 000 for a State-wide improvement of roads in Illinois.

Mr. Marr had made a special study of municipal paving and country highways. He was widely recognized as an expert of the highest class in those lines, and his advice was in constant demand. He had made a magnificent record in public office, and his re-appointment to the position by Governor Lowden was recognition of this fact. He was widely known in Illinois and adjoining States for his splendid technical and executive ability, and was in continuous request as a speaker at Good Roads conventions and similar movements. The volume of work called for by his position was so large, and the demands on his time were so great, that he literally worked night and day. Though of good physique, he was of a nervous temperament, and the net result was a break-down. For months he had been failing, but was so interested in his work that he refused to quit. He died on October 3d, 1917, and is survived by a widow and four children.

This is a brief outline of Mr. Marr's professional life; but it does scant justice to the man. Great-hearted, tolerant, and charitable Walter

* Memoir prepared by P. E. Green, M. Am. Soc. C. E.

Marr! He was always helping some one, and never asking for a return; wise in the ways of the world, wonderfully intuitive, yet excusing and condoning the little mistakes and frailties of his fellow-men.

He had a keen, analytical mind, great imagination, and bold initiative, combined with ability to execute his ideas. He was endowed. further, with that gift, supposedly so rare among engineers, of ability to appreciate artistic things. He was a painter of no mean ability, and the writer, who had been his friend and associate for many years, wondered at times that he should have become an engineer and not an artist.

His funeral was almost a sermon in itself. Men nationally known were there, as were many of those more humble ones whom he had helped generously, but always secretly. It was only when his personal effects and memoranda were searched that some real idea was gained of his charity. There were literally dozens whom he had helped financially and otherwise, about whom no one knew, and the writer who used to wonder how it happened that, with his plain habits, he was so often "broke", understood at last.

He was a member of the Illinois Society of Engineers and Surveyors and of the Springfield Engineers' Society.

Mr. Marr was elected a Member of the American Society of Civil Engineers on February 2d, 1909.

ALFRED BOARDMAN MAYHEW, M. Am. Soc. C. E.*

Died May 12th, 1918.

Alfred Boardman Mayhew was born at Heath, Mass., on September 3d, 1878. He was graduated from Tufts College in 1904, with the degree of B. S. in Civil Engineering.

His earliest engineering work was on the construction of two dams for power development, one at Gavins Fall, N. H., and one at Dorchester, Mass.

In June, 1905, Mr. Mayhew entered the employ of the United States Reclamation Service, and occupied various responsible positions connected with the design and construction of large irrigation projects in the West, principally the Minidoka and the Boise Projects in Idaho. He was Resident Engineer on the construction of several important sections of main canal, including diversion dams, bridges, and other large structures providing for cross-drainage, railroad crossings, lateral outlets, etc.

When the construction of the Arrowrock Dam (the highest dam in the world) was authorized, Mr. Mayhew was selected as Principal Engineering Assistant to the Construction Engineer, and had direct charge of the field engineering and designs. He was personally responsible for many of the details of the design of that dam and related structures, and was closely identified with the construction until it was nearly finished.

In November, 1914, Mr. Mayhew was given a furlough by the Reclamation Service, in order that he might accept a position with The Morgan Engineering Company, at Dayton, Ohio, on studies and plans for flood-prevention works in the Miami Valley, and he continued on that work as an employee of The Miami Conservancy District, after the formation of the District under the Conservancy Act of Ohio. He took a prominent part in the preparation of the designs, specifications, and estimates for the whole project, and, when construction started, was given charge of the Germantown Dam, as Division Engineer. It was while directing the work of raising a suspension bridge above the reach of flood-waters, at this dam, that Mr. Mayhew was thrown into the river and drowned on May 12th, 1918.

Mr. Mayhew's fine personality, coupled with his well-balanced capabilities as an engineer, gained for him the respect and confidence of all

* Memoir prepared by Charles H. Paul and F. C. Horn, Members, Am. Soc. C. E.

with whom he was associated. He was not only a good engineer, but also a good citizen, and took a lively interest in the affairs of the day.

He was married, on September 14th, 1911, to Miss Maude O. Greene, of Princeton, Ill., who survives him.

Mr. Mayhew was elected an Associate Member of the American Society of Civil Engineers, on November 8th, 1909, and a Member on January 18th, 1916.

MARSHALL POPE ROBERTSON, M. Am. Soc. C. E.*

Died November 2d, 1917.

Marshall Pope Robertson was born at Baton Rouge, La., on April 26th, 1861. He received his early education in the Parish of East Baton Rouge, and was graduated, in 1882, from the Louisiana State University and the Agricultural and Mechanical College, at Baton Rouge, La.

During his summer vacations in 1881 and 1882, Mr. Robertson was employed as Rodman and Resident Engineer on construction, on the Texas and Pacific Railroad.

From 1883 to 1894, he served as City Surveyor of Baton Rouge, La. While in this position, he was also engaged on work for the United States Government, as Engineer in charge of the construction of roadways to the Military Cemeteries at Port Hudson, La., Corinth, Miss., and Marietta, Ga., from 1889 to 1891, and as United States Surveyor for the Lower Texas District, on levees from Vicksburg, Miss., to the mouth of the Red River.

In August, 1896, he was appointed an Assistant to the Board of State Engineers of Louisiana, which position he retained until August, 1900, when he was appointed Assistant State Engineer and Member of the Board of State Engineers. From 1900 to 1902, Mr. Robertson had charge, for the State, of levees and canals on the Red River and, from 1902 to 1908, he was in direct charge of work on the Mississippi River and on levees on the Arkansas and Mississippi Rivers.

From 1908 to 1912, he was engaged in the private practice of engineering of an extensive and varied character. In 1912, he was again appointed a Member of the Board of State Engineers and served in that capacity continuously until his death on November 2d, 1917.

Mr. Robertson was honorable, capable, and conscientious in the discharge of his duties, to which he brought to bear the faculty of a good judgment acquired by the experience of many years, and, at the time of his death, the Board of State Engineers, of which he was for so long a member, passed resolutions expressing its appreciation of his valuable services and its sorrow at the loss sustained by his family and by the State.

Mr. Robertson was elected an Associate Member of the American Society of Civil Engineers on March 5th, 1902, and a Member on September 4th, 1906. He was also a Charter Member of the Louisiana Engineering Society.

* Memoir prepared by the Secretary from information on file at Society Headquarters.

GRANT ROHRER, M. Am. Soc. C. E.*

Died December 12th, 1916.

Grant Rohrer, son of Major Jeremiah Rohrer and Mary Redsecker Rohrer, was born in Lancaster, Pa., on October 18th, 1864. His family was of Swiss origin, and settled in Pennsylvania about 1732.

Mr. Rohrer's early education was received at Lancaster; he was graduated from the High School in 1880, and spent the next five years reading law and in miscellaneous studies.

His first engineering work was on the construction of two Western railroads: First in the Engineering Department of the Kansas, Nebraska and Dakota Railroad, and then as an Assistant in the Construction Department of the Kansas City, Wyandotte and Northwestern Railroad.

In May, 1888, he was appointed Assistant Superintendent of Construction on the Abrams Creek Branch of the West Virginia Central and Pittsburgh Railway, now the Western Maryland Railroad. This branch was a difficult piece of heavy mountainous construction connecting the Big Vein Coal Mines at Elk Garden, W. Va., with the main line in order to supplant a long inclined plane. The necessary grades exceeded 3%, and there were numerous 16° curves and a switch-back.

Mr. Rohrer and the writer were thrown together a great deal on this construction, and were room mates at Elk Garden. The business and personal association between them lasted until Mr. Rohrer's death, nearly 30 years later, and had a very material influence on the lives of both.

On the completion of the Elk Garden work and some mine surveys in that vicinity, Mr. Rohrer went to Alabama to examine and report on some coal and iron ore lands.

In 1890 he was Assistant Superintendent of Construction on 30 miles of the Ohio Extension of the Norfolk and Western Railroad until its completion.

He then returned to West Virginia to become Superintendent of the H. G. Davis Coal and Coke Company, at Thomas, W. Va., which mines were under the same interests as those that controlled the Elk Garden work he had done 5 years previously.

In 1897 he became Superintendent of the New York Office of R. H. Hood, Engineer and Contractor. This office was established by Mr. Rohrer and was very successful.

In 1899, he and the writer went to London to investigate the advisability of opening an office there, but reached the conclusion that

* Memoir prepared by R. H. Hood, M. Am. Soc. C. E.

the Boer War would be the cause of an extended depression in that market.

During the greater part of the last 20 years of his life, Mr. Rohrer was located in New York City where he built many structures and carried on a successful contracting business. He was also interested in several lines of manufacturing.

Mr. Rohrer never married. Three brothers and two sisters survive him. His closest business associates were Charles Meads and Allen N. Spooner, Members, Am. Soc. C. E., Mr. W. G. Cooper, and Dr. E. W. Caldwell.

He and the writer made tenders on the Roosevelt Dam in Arizona, the anchorages and superstructure of the Manhattan Bridge, the Narrows Siphon, for the Board of Water Supply, City of New York, and other large undertakings.

At the time of his death Mr. Rohrer was a Director and officer of numerous corporations, some of which were the R. H. Hood Company, Charles Meads and Company, Allen N. Spooner and Son, Inc., F. G. Fearon Company, Homer Corporation, etc.

He took the keenest interest in politics, and although an ardent Republican, was broad in his views and actions. His bright and cheery disposition and droll wit won him popularity among his associates, and a large portion of the later years of his life was devoted to the interests of his numerous friends. He had a pleasing personality, possessed great tact and diplomacy, and seemed to derive much pleasure from the help he extended to others.

His death occurred suddenly on December 12th, 1916, from heart disease while sitting in his office chair, and was a great surprise to his relatives and friends, as he had never shown any symptoms of disease and had always been particularly free from sickness. He was buried with Masonic rites at Lancaster, Pa.

At the time of his death he was a member of the following Societies: Pennsylvania Society, of New York, Lodge No. 43, F. and A. M., Lancaster, Pa., Chapter No. 43, R. A. M., Lancaster, Pa., Lancaster Commandery No. 13, K. T., Lancaster, Pa., and Zembo Temple, A. A. O. N. M. S., Harrisburg, Pa.

Mr. Rohrer was elected a Member of the American Society of Civil Engineers on July 1st, 1909.

EDWIN AUGUSTUS STEVENS, M. Am. Soc. C. E.*

DIED MARCH 8TH, 1918.

Edwin Augustus Stevens, head of the Stevens family of Castle Point, Hoboken, N. J., famous for its administrative and engineering achievements during a century and a quarter of continued activity, and a son of the founder of Stevens Institute of Technology, died on March 8th, 1918, of pneumonia, at Washington, D. C., where he was serving as a field officer of the Emergency Fleet Corporation.

Edwin Augustus Stevens was born in Philadelphia, Pa., on March 14th, 1858. He was educated at St. Paul's School, Concord, N. H., and at Princeton University, from which he was graduated in 1879.

He entered the employ of the Hoboken Land and Improvement Company which controlled the interests of the Stevens family in Hoboken, and, in 1885, was elected President, which office he held until his death. He was also President of the Hoboken Ferry Company, operating ferries from Hoboken to Barclay, Christopher, 14th and 23d Streets, in New York City, from 1885 to 1896, when the company was sold to other interests and later acquired by the Delaware, Lackawanna and Western Railroad, by which the ferries are now operated. He was also a Director of the First National Bank of Hoboken, the Hudson Trust Company, the Commercial Trust Company, and of the Niagara Fire Insurance Company.

Descended from a family of pioneer engineers—his grandfather, John Stevens, having invented and built a steamboat that operated on the Hudson River in 1804, and a locomotive that operated on a track in Hoboken in 1826; his uncle, Robert L. Stevens, having invented, in 1830, the present form of T-rail used in railroad track construction and having entered into a contract with the United States Government, in 1842, for the construction of the first armor-clad "war steamer"; and his father having invented, in 1832, the air-tight fireroom used in warships and in some merchant ships—Edwin Augustus Stevens had a strong inclination for engineering, and devoted much of his available time to technical subjects. Early in his career, he engaged in engineering work connected with the Hackensack Water Company, supplying the City of Hoboken. He was next interested in the development of high-speed engines and machine tools, and, later, achieved notable distinction in the design of the ferry-boat *Bergen,* built in 1889. This was the first ferry-boat to be fitted with screw propellers. It marked a signal departure in the construction of such vessels and proved so satisfactory that it has been generally adopted in place of the paddle-wheel construction, in this class of service. Subsequently,

* Memoir prepared by Professor F. DeR. Furman, Stevens Institute, Hoboken, N. J.

Mr. Stevens was engaged as Consulting Engineer by the City of New York in the matter of the construction of the Staten Island ferry-boats, and was also in charge of the design of ferry-boats for the Delaware, Lackawanna and Western Railroad Company. He was a member of the firm of Cox and Stevens, of New York City, Naval Architects and Marine Engineers, from 1905 to 1909. In 1905, Stevens Institute of Technology, of which he was a Trustee at the time of his death, conferred on him the honorary degree of Doctor of Engineering.

He was also active in civic and State matters, having served as Park Commissioner, Tax Commissioner, and as a member of the Commission which settled the long-standing dispute regarding the boundary between New Jersey and New York. He was also Chairman of the New Jersey Commission of the Interstate Palisades Park Commission. He was a Democrat in politics and served as Presidential elector in the campaigns of 1888 and 1892, when President Cleveland was the Democratic candidate.

Mr. Stevens became Adjutant of the old Ninth Regiment of the New Jersey National Guard in 1880, served later on the staff of Governor Ludlow, and, in 1884, became Colonel of the old Second Regiment. He retained this command for eight years. In 1911, President Wilson, then Governor of New Jersey, appointed him Commissioner of Highways for the State. He was re-appointed by Governor Fielder in 1914 and served in this capacity for a total of six years, when the State laws were revised and the Road Department was placed under a Highway Commission composed of eight men, of which Col. Stevens was one of the original appointees and one of the members at the time of his death.

With all his activities, he was a devoted Churchman, having served as a Vestryman of the Church of the Holy Innocents in Hoboken, Treasurer of the Protestant Episcopal Diocese of Newark, and Trustee of the Episcopal Fund and Diocesan Properties. He was also frequently in attendance as a delegate to the General Convention of the Protestant Episcopal Church.

Col. Stevens was married to Emily C. Lewis, who survives him, together with seven children: John, Edwin A., Jr., Washington L., Bayard, Basil M., Lawrence L., and Emily Lewis Stevens. Of these, Edwin A. Stevens, Jr., is a naval architect and marine engineer and is at present with the United States Shipping Board, Emergency Fleet Corporation. He has also been appointed to fill the vacancy left by his father as member of the Board of Trustees of Stevens Institute of Technology. Two other sons are in Government service, Washington Lewis Stevens as a Lieutenant in the National Army, and Basil M. Stevens as a Lieutenant in the Officers Reserve Corps.

Col. Stevens was associated with numerous organizations, always serving prominently as presiding officer, author, or active member.

He was one of the Founders of the Society of Naval Architects and Marine Engineers, and, later, served as a Vice-President; President of the American Road Builders Association; member of the Institution of Naval Architects; Life Associate of the American Society of Mechanical Engineers; member of the American Society of Naval Engineers; American Society for Testing Materials; Army League of the United States; National Marine League of the United States of America; Engineers' Club of New York City; Engineers' Club of Trenton; National Geographic Society; Historical Society of Hudson County; Princeton Club of New York; Essex Fox Hounds Riding Club; and Somerset Hills Country Club.

Among the contributions of Col. Stevens to technical literature are the following: "Performances of the *Bergen* and *Orange* Steam Ferry-Boats",* written in conjunction with Professor J. E. Denton; "Screw Ferry-Boats";† "Ferry-Boat Performances";‡ "Tidal Corrections",§ written in conjunction with Mr. C. P. Paulding; "Application of Taylor's Analysis to the Performance of the Ferry-Boat *Cincinnati*,"‖ written in conjunction with Mr. Paulding; "Progressive Trials of the Ferry-Boat *Edgewater*,"¶ written in conjunction with Mr. Paulding; "Progressive Trials of the Ferry-Boat *Bremen*";** "Screw Ferry-Boats";†† "American Competition";‡‡ "The Future of Good Roads in State and Nation".§§

Col. Stevens was elected a Member of the American Society of Civil Engineers on June 3d, 1915.

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* *Transactions*, Am. Soc. Mech. Engrs., Vol. 11 (1890), p. 372.

† *Transactions*, Soc. of Naval Archts. and Marine Engrs., Vol. 1 (1893), p. 192.

‡ *Stevens Institute Indicator*, 1900.

§ *Transactions*, Soc. of Nav. Archts. and Marine Engrs., Vol. 9 (1901), p. 235.

‖ *Stevens Institute Indicator*, 1901.

¶ *Transactions*, Soc. of Nav. Archts. and Marine Engrs., Vol. 10 (1902), p. 15.

** *Transactions*, Soc. of Nav. Archts. and Marine Engrs. Vol. 11 (1903), p. 1.

†† *Cassier's Magazine*, Vol. 6 (1894), p. 275.

‡‡ *Engineering* (London), 1899.

§§ *Scribner's Magazine*, February, 1916.

JOHN EDWARD SWANKER, M. Am. Soc. C. E.*

Died October 20th, 1917.

John Edward Swanker, the son of Augustus F. and Louisa Swanker, was born at Schenectady, N. Y., on July 4th, 1865. He received his early education in the schools of Schenectady, and then entered Union University, from which he was graduated in the class of 1887, in the Department of Civil Engineering.

Immediately after his graduation, Mr. Swanker entered the employ of the Rochester Bridge and Iron Works, of Rochester, N. Y., where he remained for about eight years. While at Rochester, he was engaged in drafting, designing, and estimating, and also had considerable experience in bridge shop practice and management, thus gaining much of that complete knowledge of American bridge building methods which has marked the career of many of the men who began their engineering experience under the late John F. Alden, M. Am. Soc. C. E.

Later, this association was continued in the Engineering Department of the Hilton Bridge and Construction Company, of Albany, N. Y., where Mr. Swanker remained until the plant was closed by the American Bridge Company, which had acquired it, in 1900. While with the Hilton Bridge and Construction Company, he was engaged on many of the larger bridges built by that company, and also had experience in the Sales Department. When the plant was taken over by the American Bridge Company, he became Manager and Engineer, which position he retained until the plant was closed.

About the time work was discontinued at the Hilton Plant of the American Bridge Company, Mr. Swanker was elected General Manager of the Tees Side Bridge and Engineering Works, Limited, of Middlesbrough, England, and held that position for six years, during which time the business was re-organized in accordance with American methods and the output doubled, while the cost of maintenance was reduced more than one-third.

During this period, many important structures were fabricated and erected under the personal direction of Mr. Swanker, several of which elicited favorable comment in the English technical press at the time. The project of the Khushalgahr Bridge in India was considered almost impracticable, but was successfully carried through to completion. The railroad bridge over the Tees River at Stockton-on-Tees was replaced by a new structure which was floated into place without interruption of traffic; as were also the transfer bridges at the North and

* Memoir prepared by James Watt, Assoc. Am. Soc. C. E., and Walter R. Marden, M. Am. Soc. C. E.

South Shields Docks, near Newcastle-on-Tyne, under very difficult tidal and local conditions.

On the completion of his engagement with the Tees Side Bridge and Engineering Works, Limited, Mr. Swanker returned to the United States and, after a short period of rest, became Engineer for David Lupton's Sons Company, of Philadelphia, Pa. Later, he was General Manager for W. N. Kratzer and Company, of Pittsburgh, Pa., after which he entered the Engineering Department of the Ferguson Steel and Iron Company, of Buffalo, N. Y., where he remained until his death.

Mr. Swanker was quick and decisive in forming engineering judgment, and will be remembered by his business associates as a man of untiring zeal and ambition, marked with sterling integrity and uprightness of character.

He was a member of the Protestant Episcopal Church and, at the time of his death, was a member of Yonnondio Lodge No. 163, F. and A. M., of Rochester, N. Y., Temple Chapter No. 5, R. A. M., of Albany, N. Y., York Preceptory, of Newcastle-on-Tyne, England, and Ismailia Temple, A. A. O. N. M. S., of Buffalo, N. Y.

While in England, Mr. Swanker was a member of the British Iron and Steel Institute; he was also a charter member of the Efficiency Society of America.

He was married on March 25th, 1896, to Clara A. Sage, of Rochester, N. Y., who survives him.

Mr. Swanker was elected a Member of the American Society of Civil Engineers on May 4th, 1904.

JOHN DASH VAN BUREN, M. Am. Soc. C. E.*

Died March 11th, 1918.

John Dash Van Buren, the son of John Dash and Elvira (Aymar) Van Buren, was born in New York City, on August 8th, 1838. He studied Civil Engineering at Lawrence Scientific School of Harvard University and Rensselaer Polytechnic Institute, Troy, N. Y., and was graduated from the latter in 1860, with the degree of Civil Engineer.

Mr. Van Buren began engineering work under the late Alfred Wingate Craven, Past-President, Am. Soc. C. E., then Chief Engineer of the Croton Aqueduct Department, but resigned when the Civil War broke out in 1861 to enter the Navy as an Assistant Engineer with the rank of Lieutenant. As such, he served with the Gulf Squadron in the James River Campaign and in the Flying Squadron under Commander Wilkes. He was also with the squadron which formed part of the forces under Gen. McClellan in the battle of Malvern Hill.

Following this, Mr. Van Buren was detailed for special duty as Assistant Professor of Natural Philosophy and Engineering at the United States Naval Academy where he served for four years. During the next two years he was detailed as Assistant to Commander Isherwood, at the Bureau of Engineering of the Navy Department. In 1865, he was promoted to be First Assistant Engineer, serving as such until 1868 when he resigned his commission and returned to civil life.

In 1869, Mr. Van Buren was admitted to the bar of New York City. He practiced law only a short time, however, and gave it up to return to his former profession as a Civil Engineer.

From 1870 to 1875, he served as Assistant to Generals McClellan and Graham in the Department of Docks of New York City, having been engaged on the reconstruction of the water front.

In 1875, Mr. Van Buren was appointed by Governor Tilden as a member of a commission to investigate the canals of New York State. As a result of his work on this Commission, he was elected to the office of State Engineer and Surveyor, serving in that capacity from 1876 to 1878, during which time many important reforms were carried out in that Department.

On the expiration of his term of office as State Engineer, Mr. Van Buren again took up the private practice of engineering, and made his home at Newburgh, N. Y., where he was employed on the extension of the water-works. As a Consulting Engineer he was engaged on

* Memoir prepared by the Secretary from information furnished by Robert Van Buren, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

many important engineering works until his retirement from active life several years ago. For a number of years he had made his home at New Brighton, Staten Island, where his death occurred on March 11th, 1918.

Mr. Van Buren was a gentleman of the old school. He had lived a quiet retired life, but was a very congenial companion, and stood high as an Engineer, having been considered one of the best mathematicians in the Profession.

He was a frequent contributor to the technical press and to the publications of the Society, having written papers on "Quay and Other Retaining Walls",* "The Improvement of the Water Front of the City of New York",† and "Notes on High Masonry Dams",‡ as well as discussions on papers by other members.

Mr. Van Buren was married on November 24th, 1875, to Miss Elizabeth Ludlow Jones, of New York City. They had two sons, one of whom died several years ago; the other is now serving in an Engineer Regiment with the American Expeditionary Forces.

He was a Democrat in politics, and was a member of the Loyal Legion, American Geographical Society, Society of Naval Engineers, Society of Naval Architects and Marine Engineers, the Holland Society, and the St. Nicholas Society.

Mr. Van Buren was elected a Member of the American Society of Civil Engineers on May 20th, 1868.

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* *Transactions*, Am. Soc. C. E., Vol. II (1873), p. 193.

† *Transactions*, Am. Soc. C. E., Vol. III (1874), p. 172.

‡ *Transactions*, Am. Soc. C. E., Vol. XXXIV (1895), p. 493.

CLARENCE BROWNING VORCE, M. Am. Soc. C. E.*

DIED FEBRUARY 4TH, 1918.

Clarence Browning Vorce was born on June 23d, 1866, in New York City, but during his boyhood lived at Farmington, Conn. He received his technical education at the Massachusetts Institute of Technology, from which he was graduated in 1888, receiving the degree of Bachelor of Science.

In July, 1889, he joined the Lake Shore and Michigan Southern Railroad and remained with that Company until February, 1893, holding successively the positions of Transitman, Assistant Engineer, and Principal Assistant Engineer. Thereafter, he was Assistant Engineer to the Engineer of Construction of the New York, New Haven and Hartford Railroad, on four-track improvements near New York City until 1897. From 1897 until 1906 he was in private practice at Hartford, Conn., engaged in general engineering work.

In 1906, Mr. Vorce entered the employ of Sanderson and Porter, Engineers, and was thereafter actively connected with important railway and hydraulic construction in the New York and San Francisco offices of that firm. Later, he entered the employ of the British Columbia Electric Railway Company as Engineer of Railways, in which position he had charge of all new construction and of the maintenance of all lines, making a notable record in design and in rapid economical execution of track work in paved districts, involving large expenditure.

He left this work to re-enter the employ of Sanderson and Porter, and was associated continuously with the work of the New York office from 1914 until his death, which occurred at Roosevelt Hospital, in New York City, on February 4th, 1918, after an operation from the effects of which he failed to rally.

All who knew Mr. Vorce and his work considered him to be an engineer of exceptional ability and sound judgment. He was a member of the Electric Railway Association and Past-President and member of the Connecticut Society of Civil Engineers.

In 1892, he was married to Miss Virginia Osborn, of New Haven, Conn., who, with a daughter, survives him.

Mr. Vorce was elected a Junior of the American Society of Civil Engineers on April 30th, 1895; an Associate Member on October 7th, 1896; and a Member on May 2d, 1900.

* Memoir prepared by friends of Mr. Vorce in the office of Sanderson and Porter, Engineers, New York City.

BERNARD MATTHEW WAGNER, M. Am. Soc. C. E.*

Died June 15th, 1918.

Bernard Matthew Wagner, the son of Bernhard and Pauline Wagner, was born on September 9th, 1868, at St. Gall, Switzerland. He was brought to the United States as a child, and was educated in the public schools of New York City, taking the degree of Bachelor of Science at Cooper Institute in 1888. Thereafter, he returned to Switzerland, and entered the Swiss Federal Polytechnic, at Zurich, from which he was graduated as Ingenieur in 1892. He then became Assistant to M. E. Paschaud, Engineer of the Swiss Occidental Railroad, and was engaged in survey and office work on that system. Later, he was Assistant to the Chief Engineer of the Grand Ducal Railroad, in which position he was engaged on bridge and construction work. Mr. Wagner also served for a short time as Draftsman in the Raub Locomotive Works.

In 1894, he returned to the United States and, in 1895, entered the service of the Department of Water Supply of New York City. He was appointed Assistant to I. M. de Varona, M. Am. Soc. C. E., Chief Engineer of the Brooklyn Water Department, and continued in the employ of the city until 1912.

During his seventeen years of municipal service, Mr. Wagner was engaged on the important and extensive construction work carried out by Mr. de Varona, notably the large steel main extending from Jamaica to Ridgewood, planning and drawing specifications for mains and standpipe work, and designing and erecting new steam-generating plants and pumping plants at Mount Prospect and Baldwin, Long Island. Later, he was placed in charge of filter-plant construction and maintenance, and supervised the planning and construction of the mechanical filter plants at Jameco, and Springfield, Long Island.

After leaving the service of the city, Mr. Wagner became associated with the R. P. Bolton Company, and took part in the preparation of the Citizens' Plan for the New York Central West Side Improvement, and in plans for power and hydro-electric machinery.

On the outbreak of the Spanish War in 1898, Mr. Wagner was a member of the Twelfth Regiment, New York National Guard, and, during his service with that regiment in the war, he did much effective work as a field officer, in which capacity his ability as a civil engineer was of special value in connection with cantonment details. He left the service with the rank of Captain.

On the entry of the United States into the present war, Captain Wagner immediately volunteered his services in the Reserve Corps, and

* Memoir prepared by Reginald Pelham Bolton, M. Am. Soc. C. E.

was given a commission as Major. His health, however, precluded his taking part in active service, and in the winter of 1917, he was compelled to resign his commission. He thereupon devoted himself to shipyard and ship planning, in which he was engaged at the time of his death.

Major Wagner, whose amiable character, high integrity, and great engineering ability had endeared him to his employers, his associates, and to his wide circle of friends, had made his home in Brooklyn, N. Y., for many years. He was married there in 1898, and his body has been laid to rest in Greenwood Cemetery. He is survived by his widow and one daughter.

Major Wagner was elected an Associate Member of the American Society of Civil Engineers on May 3d, 1899, and a Member on December 1st, 1903.

JOHN WATERHOUSE, M. Am. Soc. C. E.*

Died February 20th, 1918.

John Waterhouse was born in Bucks County, Pa., on October 4th, 1839. His father, Ingham Waterhouse, came from Kingwood, N. J., where the family had settled on arriving from England. His mother, Frances Calvin Waterhouse, who was born in Bucks County, Pa., was a daughter of Joshua B. Calvin, at one time a member of the Pennsylvania Legislature, and a direct descendant of John Calvin the reformer. Another ancestor, Dr. Benjamin Waterhouse, who was born in Newport, R. I., was a prominent surgeon in the Continental Army, and was said to have been the physician who introduced vaccination in America. He taught in Harvard College from 1783 to 1812, as Hersey Professor in theory and practice of physics.

At the age of eleven, John Waterhouse was a boat boy on the Delaware Division of the Pennsylvania Canal. Later, during his school years, he worked on a farm, and was employed for four years in mercantile work at Lambertville, N. J., until he was twenty. On the breaking out of the Civil War, he enlisted in the Third Regiment of New Jersey, and, on the completion of his three months' term of enlistment, went into business with his brother in Trenton, where he remained about a year.

Mr. Waterhouse began his engineering career in 1863 when he went to Pittsburgh, Pa., and joined the engineering force of the Pittsburg and Steubenville Section of the Pennsylvania Railroad as an Axeman and Rodman, being transferred later to the Lewiston Branch of the same railroad. He spent some time in Oil City, Pa., soon after the outbreak of the oil fever there, as a Surveyor, but illness obliged him to give up these activities, and he went West, entering the service of the Missouri Pacific Railroad as a Transitman. From 1865 to 1867, he was Assistant Engineer of that railroad at Kansas City, and Principal Assistant Engineer on the Kansas City and Camaron Railroad. During 1867 and 1868, he was in charge of the Tunnel Division of the west shore, North Missouri Railroad, at Liberty Landing, Mo. From 1868 to 1869, he was employed as Division Engineer with the Kansas City, Fort Scott and Gulf Railroad, at Paola, Kans., and, in 1869 and 1870, as Principal Assistant on surveys through Indian Territory for the same railroad. From then until the latter part of 1872, he was at Lebanon, Mo., as Principal Assistant Engineer for the Laclede and Fort Scott Railroad. During 1872 and 1873 he made surveys for, and located the railroad connection to, the Breen Iron Mine, southwest of Escanaba, Mich., for the Chicago and Northwest-

* Memoir prepared by H. I. Latimer, Esq., and Robert Ridgway, M. Am. Soc. C. E.

ern Railway. Following this, he was in the service of the United States Government until 1874, on surveys of the Mississippi and Osage Rivers, with station at St. Louis, Mo., and, from 1874 to 1876, was Chief Engineer of the Paris and Danville Railroad in Illinois.

In 1877, Mr. Waterhouse came to New York City, and, on the recommendation of William F. Shunk, the Chief Engineer, he accepted an appointment as Assistant Engineer with the New York Loan and Improvement Company, which company was then constructing the elevated railroad on Sixth Avenue for the Metropolitan Elevated Railway Company. The Sixth Avenue line was one of four lines of rapid transit elevated railroads then under construction in the city, the others being known as the Second Avenue, Third Avenue, and Ninth Avenue lines. The Second Avenue and Sixth Avenue lines were controlled by the Metropolitan Company, the other two by the New York Elevated Railroad Company, but on September 1st, 1879, all these properties were taken over for operation by the Manhattan Railway Company. Mr. Waterhouse resigned his position with the latter Company on July 1st, 1881, but was re-appointed Assistant Engineer on May 13th, 1882. Thereafter, he continued in its service until March 1st, 1899. On August 1st, 1885, he was promoted to the grade of Principal Assistant Engineer, and on March 1st, 1890, was made Chief Engineer of the Company. In January, 1899, he retired as Chief Engineer, and on January 23d, he was made Consulting Engineer, resigning this position on March 1st, 1899, after practically 22 years of service on the construction and maintenance of this extensive system of elevated railroads, about one-half of their life. The records of the Chief Engineer's office are replete with evidences of his conscientious personal attention to details and of his skill as an Engineer. At the time of his retirement as Chief Engineer, the system operated by the Manhattan Railway Company, comprised 32 miles of elevated railroad structures, or 107 miles of single track. This system included not only the original four lines on Manhattan Island, but the connecting line north of the Harlem River in what is now the Borough of the Bronx, which was constructed by and leased from the Suburban Rapid Transit Company.

Mr. Waterhouse was in direct charge of the construction of the Western Division of the Metropolitan Lines extending from 83d Street on Ninth (now Columbus) Avenue to 155th Street on Eighth Avenue. This extension was built in 1878 and 1879, and included the famous 110th Street reverse curve, the highest elevated railway structure for urban service in the world, its maximum height from the street surface to the base of rail being 60 ft. Later, as Chief Engineer, he personally designed the present four-track structure on West Broadway between Chambers and Franklin Streets, known as the "West Broadway Pocket." The design of this pocket was discussed and criticized

at the time by a number of engineers and architects, but was built according to Mr. Waterhouse's plan. The fact that it still remains in use after nearly thirty years of constant service indicates the designer's good judgment and skill.

Mr. Waterhouse was always methodical and simple in his tastes and habits, and if it could be said that he had any hobbies, they were good books, mathematics, and inventions. Some years ago, he originated a method for proving the Pythagorean theorem in geometry, which was published in the *Scientific American,* and which interested instructors of geometry all over the country, bringing many letters of commendation to him from prominent teachers. Faithful to his duties and loyal to his superiors, he enjoyed the respect as well as the affection of his many subordinates. Perhaps he was lacking in aggressiveness, but in his gentle, quiet way he was convincing, and the friends he made were life-long. Those who knew him loved him, and, to his intimates, he was regarded as one of God's noblemen.

More than ten years ago Mr. Waterhouse became totally blind and deaf, but he bore his terrible affliction with the characteristic cheerfulness for which he was noted, occupying much of his time in reading books for the blind, the intricacies of which he mastered within a week and without instruction. During this period he invented a very ingenious writing board which enabled the user, by the sense of touch, to write in a natural hand, keep to straight lines, observe the required spaces between them, number the sheets as finished, and record them in consecutive order. Mr. Waterhouse used this board for several years with great success.

In 1872, he was married to Miss Letitia Southwick, who died in 1873. He is survived by his daughter, Miss Pearl Frances Waterhouse, of New York City. The bond between father and daughter was an unusually close one, and she was his constant companion during the years of his affliction.

Mr. Waterhouse was elected a Member of the American Society of Civil Engineers on October 4th, 1899.

AUGUSTINE WASHINGTON WRIGHT, M. Am. Soc. C. E.*

Died February 3d, 1918.

Augustine Washington Wright, the son of John Stephen and Catharine Blackburn (Turner) Wright, was born in Chicago, Ill., on May 29th, 1847, his father having been one of the pioneer settlers of that city.

After his preliminary education at the public schools, Mr. Wright studied mathematics with a private tutor. In April, 1867, he began his engineering work on the Eastern Division of the Union Pacific Railway, and, in 1867 and 1868, was employed in various capacities on the surveys for that road across the plains of Kansas, Nebraska, and Colorado.

During 1869, he was engaged as Division Engineer on the location and construction of the Lexington and St. Louis Railroad, and, in 1870 and a part of 1871, he was connected with the South Branch of the Louisiana and Missouri River Railway in the same capacity.

In the latter part of 1871, Mr. Wright was appointed Assistant Engineer on the Arkansas Central Railway and, in 1872, he served as Chief Engineer of the same road.

In 1873, he returned to Chicago, Ill., as Engineer and Superintendent of the Ransome Stone Works, which position he held until 1875, when he was appointed Chief Engineer of the Federal Creek Valley Railroad. He also served, in 1876, as Chief Engineer of the St. Louis and Toledo Air Line Railroad.

In the latter part of 1877, Mr. Wright became Chief Engineer and General Superintendent of the Havana, Rantoul, and Eastern Railway. He had become interested in steel cable railroads, however, and, in 1878, he resigned this position, to accept that of Chief Engineer and Superintendent of Track and Construction of the North Chicago City Railway, the cable system established in Chicago by the late Charles T. Yerkes.

Mr. Wright remained with the North Chicago City Railway for a number of years. In 1889 and 1890, he built the first cable railroad in Los Angeles, Cal., and, at the same time, he had two cable lines under construction in St. Louis, Mo. He also acted as Consulting Engineer in connection with the construction of first traction railway in New York City. When Mr. Wright retired from active railroad work in 1890, he had become such an authority on traction work that he was called in to advise in a consulting capacity on such construction in all parts of the United States.

* Memoir prepared by the Secretary from information furnished by Paul Shoup, Esq., and on file at the Headquarters of the Society.

In 1899, Mr. Wright gave up his consulting practice in Chicago, Ill., and settled with his family at Pomona, Cal., where he became a rancher on a large scale. In 1912, however, he sold his property at Pomona and removed to Los Angeles, where, on June 24th, 1913, he was appointed a member of the Board of Public Utilities. On January 4th, 1915, he was re-appointed to succeed himself, and served as a member of the Board until January 1st, 1918, when he resigned on account of ill health. From December 17th, 1913, to January 27th, 1915, he had served as President of the Board.

Mr. Wright was one of the foremost traction men in the United States and an able engineer of wide experience. He contributed articles on street car railroads to *The Scientific American, Railroad Journal, Engineering News, Railway Gazette, Journal of Railway Appliances, Street Railways, Popular Science News,* and *American Engineer.* Some of these were compiled later in "American Street Railways." A review of this book states that it displays not only intimate and minute acquaintance with all engineering and constructive questions, but literary ability of a very high order.

Mr. Wright's first wife, Miss Kate Hanly, to whom he was married in 1874, died in 1877. He died at his home in Los Angeles, Cal., on February 3d, 1918, and is survived by his widow, who was Miss Natalie Jordan, of St. Louis, Mo., to whom he was married on January 1st, 1890, and by two daughters and a son, the latter a student at Stanford University.

Mr. Wright was elected a Member of the American Society of Civil Engineers on May 5th, 1886. He was also a member of the Western Society of Engineers, having served as its President in 1886-87.

JOHN HATFIELD FRAZEE, Assoc. M. Am. Soc. C. E.*

Died May 4th, 1917.

John Hatfield Frazee, the son of Lawrence Fisher Frazee and Sarah Lee (Stark) Frazee, was born at South Amboy, N. J., on September 23d, 1867. His father, who was also born in New Jersey, was engaged in the railroad and transportation business, and, during the Civil War, had charge of transporting troops for the Federal Government. He devoted much of his time to various inventions, including a lifeboat, which was adopted by the Navy Department, and a canalboat which was designed to avoid the destruction of banks by wave action; he also invented the collapsible gates commonly used on elevated railway and subway trains and on ferry-boats, and applied for and secured patents on various devices for improving transportation facilities.

His son, John Hatfield Frazee, received his engineering degree from New York University, from which he was graduated in June, 1886. Shortly after, he entered the office of the late Charles B. Brush, M. Am. Soc. C. E., being assigned by Mr. Brush to general city work. Entering the service of the Pennsylvania Railroad during the summer of 1887, he served in the field party of E. F. Brooks, Engineer of Maintenance of Way, on the Princeton Draw-bridge, the new passenger line through Elizabeth, N. J., and on a number of bridge and grade changes, including the Jersey City elevated and terminal work. Mr. Frazee was then appointed Assistant Engineer in charge of the location and construction of the Jersey City elevated and terminal work, and, on its completion, had charge of the Harsimus Cove third-track improvement, including four bridges and various improvements in and about the Jersey meadows and river crossings. He was then given charge of the elevated work contemplated through Elizabeth, including the location and construction of abutments, piers, ribbed arch bridges, retaining walls, and about 5 000 ft. of double-track trestle.

In the early part of 1893, Mr. Frazee resigned from the Pennsylvania Railroad service to become Engineer of Construction on the Lancaster and Columbia Railroad, a 12-mile line of standard construction. Afterward he had charge of the location and construction of trolley lines in and about Hartford, Conn., and, subsequently, of the location, estimates, and construction of the Ivy Hill Storage Reservoir, for Newark, N. J. In May, 1894, he was appointed Chief Engineer of the Hartford Street Railway. This System was about to be

* Memoir prepared by R. W. Creuzbaur, M. Am. Soc. C. E.

changed from horse to electric power, and the work planned and carried out involved the reconstruction of existing lines, 20 miles of new tracks, trestles, bridge abutments, and power stations, all on the best standards.

In February, 1897, Mr. Frazee entered the service of the City of New York, and his record up to the time of his death shows great activity and capacity not only for thorough study of engineering projects and contracts, but also for many constructive changes in various branches of city work. He was first engaged in 1897 as Assistant to the writer on street and highway improvements in the Department of Public Works in the old City of New York. This work cost $3 000 000 during that year, and included the reconstruction of Park Avenue over the New York Central Railroad tracks, and many street changes and improvements. He was also specially detailed at this time on studies of electrolysis of underground pipes, etc.

On the consolidation of "Greater New York" in January, 1898, the Highway Bureau of Manhattan had jurisdiction over the whole city. and the engineers proceeded to standardize specifications for such work, Mr. Frazee having an important part in this difficult undertaking. Subsequently, he resigned to engage for some time in his former specialty of electric railway construction, as Resident Engineer of the Dallas (Tex.) Consolidated Electric Street Railway. He was afterward engaged on the location and designs for the New York Connecting Railway, including the Hell Gate Bridge project.

In December, 1904, he accepted a position with the Department of Finance of New York City, reporting to the writer in the general supervision of engineering work. In the early part of the following year, he was duly appointed a First Assistant Engineer, and thereafter served in many important matters, reporting as a technical adviser on many hundreds of projects and contracts. He was advanced in five promotions until 1914, when he was transferred to the new Bureau of Contract Supervision of the Board of Estimate, for which, at the time of his death, he was handling many of the most important municipal engineering questions and contract works, including all the dual subway contract propositions for the city.

Mr. Frazee had special ability for analytical study of these matters, and, having gone through a training in design and construction, his reports on the many projects and contract work placed in his charge were gauged by the stern necessities of good practice and economy, and were of great practical and money value to the City. The writer looked to him, in these dozen years of such special service, as an expert of much ability along these lines of work, and, above all, as a man of absolute honesty and perfect candor

in his straightforward exposition of defects in work or design and his constructive recommendations on plans and methods involved in the propositions of the many departments and bureaus having authority to spend the money of the taxpayers.

His many reports on these railroad, bridge, dock, sewer, and other questions, in the works which the City has been called on to finance, indicate his ability and impartiality in handling the responsible duties assigned to him.

Referring only briefly to the most important matters on which he made such original recommendations for changes in plans and estimates prepared by the various departmental engineers, and where he contested claims connected with the many contracts made the subject of litigation and handled by the Comptroller's engineers, his recommendation to the writer of the advantage in municipal works of utilizing what was called "percentage unit bidding", is noted. This method was put into effect in the Bureau of Sewers, of Brooklyn, in 1907, where, for 10 years, it has operated to promote competition and reduce unbalanced bids, litigation, and undue profit or loss to the contractors.

By diligent investigation he also brought out cases where City funds were afterward saved in carrying out sewer and water supply projects. These are small matters compared with the saving effected by his resistance to applications for large appropriations for contracts of questionable utility to be loaded on the City Treasury.

Mr. Frazee was also selected at various times to serve on a board of engineers to report to the City authorities on difficult matters affecting designs and ways and means of constructing subways and rapid transit extensions. When considering the question of whether or not New York City should build pipe galleries in connection with its more important main lines of subways, he examined very carefully the questions of first cost, up-keep, and the off-setting savings in mutilation and maintenance of pavements, and favored pipe gallery construction on certain lines of the subway work. In 1910, he was selected by the writer, acting as chairman of a committee for the Mayor, Comptroller, and Aldermanic President, to revise the contract forms and specifications for the entire Tri-borough System, presented to the Board of Estimate by the Public Service Commission as ready for bids. Out of forty changes recommended by this committee, in which Mr. Frazee took a foremost part, twenty-eight were accepted by the Commission as recommended, and twelve were made the subject of compromise. Mr. Frazee originated most of these constructive recommendations.

With the passing away of John Frazee, the City of New York has lost an engineer who, in the writer's opinion, after thirty years of close

observation, has done more good work along original lines of engineering reform than any other man in the service. His personal characteristics were such that, although generous and open-handed to a fault among his friends and confidants, he stood out, even when unsupported, with inflexible determination in every case where his employer, the great corporation of the City of New York, had any interest.

Mr. Frazee was elected an Associate Member of the American Society of Civil Engineers on December 6th, 1899.

EDWARD McKIM HAGAR, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 18TH, 1918.

Edward McKim Hagar was born at Salem, Mass., on June 21st, 1873.

In 1893, at the age of twenty, he was graduated from Massachusetts Institute of Technology, and, a year later, completed a post-graduate course at Cornell University.

Mr. Hagar began his business career with the organization, at Chicago, Ill., of the firm of Edward M. Hagar and Company, a machinery sales concern representing the Mesta Machine Company, the Southwark Foundry and Machine Company, and others.

When the United States Steel Corporation was formed, in 1900, Mr. Hagar was made General Manager of the Cement Department of the Illinois Steel Company, at Chicago. On October 1st, 1906, the Universal Portland Cement Company, a subsidiary of the United States Steel Corporation, was organized, and he was elected its President. During his connection with the cement industry Mr. Hagar served for two years as President of the Association of American Portland Cement Manufacturers, and also organized the Cement Products Exhibition Company which held its first annual industrial exhibition in 1907.

On January 28th, 1915, he resigned the Presidency of the Universal Portland Cement Company to organize a new cement company for the purpose of acquiring and operating a chain of strategically located cement plants across the continent. He is credited with failure to finance this venture, but to the uninitiated the following sidelight is interesting. A preliminary agreement had been reached with certain New York bankers and a plan of financing approved. Mr. Hagar and his attorney were just entering the elevator of the building in which the bankers were located, with the papers drawn up and approved, ready for final signatures, when the announcement of the *Lusitania* sinking was made. The bankers said, "We had better wait until we see what this means."

About this time, Mr. Hagar was called to New York City by another group of financiers and offered the Presidency of The Wright Company, which had just acquired control of the Simplex Automobile Company for the purpose of building the remarkable "Hispano-Suiza" aeronautic motor, now well known at the front abroad. Later, The Wright Company took over the Glenn L. Martin Company of Los Angeles, Cal., aeroplane manufacturers, and the Wright-Martin Aircraft Corporation was organized and Mr. Hagar elected its President

* Memoir prepared by E. W. Burbott, Esq.

on September 8th, 1916. Here his great belief and enthusiasm in the ultimate future of the aircraft industry could not be confined within the limits set by his Board of Directors, and, on February 17th, 1917, he resigned the Presidency of that Corporation.

He next became connected with the American International Corporation, where he organized, and, on September 7th, 1917, was elected President of, the American International Steel Corporation, a subsidiary formed for the purpose of merchandising American steel and steel products abroad. Mr. Hagar retained this position until his death which occurred at his residence, 960 Park Avenue, New York City, on January 18th, 1918, of acute pneumonia, after an illness of only five days.

He was a member of the following societies and clubs: American Society of Mechanical Engineers; American Society for Testing Materials; Western Society of Civil Engineers; American Institute of Mining Engineers; Automobile Club of America; Technology Club of New York; New York Yacht Club; Engineers Club; University Club; The Recess; Duquesne Club of Pittsburgh; Chicago Club; and the Phi Kappa Psi Fraternity.

Mr. Hagar was elected an Associate Member of the American Society of Civil Engineers on February 6th, 1901.

WILLIAM HAGUE, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 2D, 1918.

William Hague, First Lieutenant, 116th Regiment of Engineers, American Expeditionary Force, died in active service, of pneumonia, on January 2d, 1918, at the Red Cross Hospital, in Paris, France.

William Hague, the only son of James D. Hague, a very distinguished mining engineer, and Mary Ward (Foote) Hague, of Guilford, Conn., was born in Orange, N. J., on March 31st, 1882. He attended Milton Academy, at Milton, Mass., and was graduated from Harvard University in the class of 1904, taking a post-graduate course in geology and mining.

On leaving Harvard, Mr. Hague went to work as a surveyor's helper in the mines of the Copper Queen Company, at Bisbee, Ariz., but was soon engaged as Assistant on the construction work of the Copper Queen smelting plant, at Douglas. In the latter part of 1905, he was transferred to the Geological Department of the Company and continued in that work until May, 1906. The summer of 1906 was spent in surveying and prospecting lands in the vicinity of the Calumet and Hecla Mines, in which his father was interested. In the fall, he returned as Assistant on the construction of the Douglas Plant of the Copper Queen Company, and remained there for more than a year. During the winter of 1908, he was traveling and studying, increasing his experience and making careful notes of interesting mines and machinery, and working occasionally. For two months, he was Night Boss in the Cyanide Plant of the Guanajuato Consolidated Mine, but his main purpose was the acquirement of knowledge and experience.

A case of rather complicated appendicitis kept Mr. Hague from work during most of 1908, but, in September, after the death of his father, he was elected Managing Director of the North Star Mines Company. As his father had been President of this Company and had operated the mines successfully for more than twenty-five years, Mr. Hague was his natural successor. During 1909 and 1910, however, he was engaged in geological work in Bisbee, Ariz., for the Copper Queen Company. He also examined all the principal copper mines of Nevada and Arizona, and his notebooks of his observations of the "Porphyry Coppers" are wonders of intelligent detail, arranged in such order and clearness that any engineer might profit by them.

In 1911, Mr. Hague was Assistant to Mr. J. R. Finlay in the appraisal of copper mines for the State of Michigan. Since 1910, when he married Elizabeth Stone, of Milton, Mass., he had resided in Grass

* Memoir prepared by Arthur DeWint Foote, M. Am. Soc. C. E.

Valley, or rather at the North Star Mine. His insatiable thirst for knowledge, however, would not allow him rest from the search, and he studied the mines of the Mother Lode to the south, and of Sierra and Plumas to the north, filling his notebooks as usual with details and maps for future use. He was not only a student of mines, but a great reader of history from his youth, and he constantly acquired the general engineering knowledge obtainable from books, societies, and periodicals. In order that he might broaden his knowledge of engineering, he became a member of the American Institute of Mining Engineers, the Mining and Metallurgical Society, and other technical associations. He was also a member of the Harvard Club, the Engineers Club, and the Downtown Association of New York.

Mr. Hague early recognized the results that would inevitably come from Germany's action in Belgium and on the sea, and prepared himself by going to Plattsburgh in 1916 to learn the trade of war. After earnest work there, he obtained a commission as First Lieutenant in the Engineers Reserve Corps, and was called into service soon after the declaration of war: first at the Presidio; thence to Vancouver Barracks, American Lake, Charlotte, Mineola, and France.

Trained to order and system, and loving them, Lieut. Hague strove for months to bring method and direction into the chaos of the new training camps and teach uncouth boys to be men and soldiers—detailed drudgery of the most wearing kind to his orderly mind. He never failed for a moment in the work, though it undoubtedly weakened him in health, so that when attacked by illness he lived only a few days.

As a brilliant editor has written:

"Military preparation is inevitably attended by some loss of life. In our case, this time, it has been comparatively low. But every faithful soldier, every zealous officer who has died in camp or in transport has given his life as truly to win the war as though he died in actual engagement with the enemy. Whoever has dedicated himself to the war has bargained to give his life whenever it shall be demanded of him. Whether or when the summons will come he cannot tell, but whenever it does come, be it by pneumonia or other camp disease, or by drowning in transport, or by death from wounds at the front, it is equally a case of dedication fulfilled and sacrifice accepted for the great cause that beckons to us all."

Young, with a wife and son, a goodly fortune, a worthy position in his profession, everything to make life worth living, and living that life worthily and happily, William Hague carefully planned and deliberately worked to give it all as the simple duty of citizenship.

Mr. Hague was elected an Associate Member of the American Society of Civil Engineers, on October 4th, 1910.

JOEL MANNING HOWARD, Assoc. M. Am. Soc. C. E.*

Died May 25th, 1917.

Joel Manning Howard was born in Ogdensburg, N. Y., on April 24th, 1880. His early education was obtained in the public schools of that city, and he was graduated from the Ogdensburg Free Academy. He then entered the Mechanical Engineering Course at Clarkson College of Technology, at Potsdam, N. Y., from which he was graduated in 1902, with the degree of B. S.

Immediately after his graduation, Mr. Howard entered the service of the United States Lake Survey, where he was engaged on the topographic and hydrographic survey of the St. Lawrence River. Upon leaving the Government service, he was employed by the Barber Asphalt Paving Company, and afterward had charge of the water-proofing of the subway tunnels in New York City, for the Sicilian Asphalt Paving Company.

In June, 1907, Mr. Howard received an appointment from the State Engineer of New York, and was assigned to take charge of contracts in Oswego County. In 1909, he was transferred to work in St. Lawrence County, and had complete charge of all the State road work of that County. He held this position until 1913, when he was appointed County Superintendent of Highways by the Board of Supervisors of St. Lawrence County, which position he held until his death.

Mr. Howard was recognized by all who knew him as a man of splendid character and high ideals, which, together with conscientious service and interest in his profession, his community, and public affairs, made him a valuable public servant.

He is survived by his wife, who was Miss Gertrude Crapser, of Waddington, N. Y., and two sons. He was a member of Acacian Lodge, F. and A. M., and the Century Club, of Ogdensburg, N. Y.

Mr. Howard was elected an Associate Member of the American Society of Civil Engineers on June 4th, 1913.

* Memoir prepared by Charles A. Pohl, M. Am. Soc. C. E.

CLARK WALLACE THOMPSON, Assoc. M. Am. Soc. C. E.*

DIED MARCH 16TH, 1918.

Clark Wallace Thompson was born at St. Paul, Minn., on August 27th, 1866. His father was the late Clark W. Thompson who was Superintendent of Indian Affairs under President Lincoln. The family moved to La Crosse, Wis., and it was here that Mr. Thompson grew to manhood. He was educated in the public schools of that city and was graduated from the High School. He then entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1887 with high honors as a Civil Engineer.

He began his engineering work in the fall of 1887 as a Draftsman with the "Soo" Railroad Company, at Minneapolis, Minn., resigning to go to the Duluth and Iron Range Railroad Company as Assistant to the Resident Engineer on construction work.

During the summer and fall of 1888 Mr. Thompson was Engineer in charge of the construction of a submerged water main connecting the Duluth and West Superior Water-Works, as well as a pump-house intake pipe and crib for the Superior Water-Works.

In 1889, he was engaged on water-works construction in North St. Paul, Minn., and also at Duluth, Minn., where he supervised and built a storage reservoir and gas-holder for the Duluth Gas and Water Company. In 1890, Mr. Thompson went to Lakeside, Minn., where he was employed on the construction of the water-works. He was also engaged on the building of gas-works for the Duluth Gas and Water Company.

In 1891, he returned to La Crosse, Wis., where he opened an office as a Consulting Engineer. Later, he became associated with the S. Y. Hyde Elevator Company, and designed and built several grain elevators. In 1893 he was elected Treasurer of that Company, which position he retained until 1901, when he moved to Cascade Locks, Ore., to undertake the management of the Wind River Lumber Company. He designed and built that Company's lumber mill, said to be one of the best on the Pacific Coast, and the structures for driving and flooding the streams to which the timber of the Company is tributary.

Mr. Thompson had been in poor health for a year or more, from heart trouble. In the fall of 1917, in the hope that the mild climate of Southern California might benefit him, he went to Los Angeles, Cal., where he died on March 16th, 1918.

* Memoir prepared by the Secretary from information on file at the Headquarters of the Society.

In 1892, Mr. Thompson was married to Miss Jessie M. Hyde, the eldest daughter of the late S. Y. Hyde, of La Crosse, Wis., who with seven children, survives him.

During his residence in La Crosse, he took an active part in public affairs, and, as a member of the City Council, had given efficient and loyal service, especially in problems in which his engineering experience was of value. He was known on the Pacific Coast as a clever engineer, and his many friends will remember him as a loyal fellow citizen, a generous host and friend, and for all the qualities that go to make a lovable character.

Mr. Thompson was elected a Junior of the American Society of Civil Engineers on March 5th, 1890, and an Associate Member on July 3d, 1895.

LOUIS WACHTEL, Assoc. M. Am. Soc. C. E.*

Died October 10th, 1917.

Louis Wachtel was born in Gloversville, N. Y., on December 9th, 1885. He spent his boyhood in Gloversville and received his early education in the public schools of that place. After graduating from the High School in 1903, he entered Union College to prepare for his future profession as a Civil Engineer. While there he was a member of the Delta Phi Fraternity.

After leaving college in 1906, Mr. Wachtel entered the employ of the New York State Highway Department as a Chainman. By his capabilities and industry, he steadily advanced through the various civil service grades until he had reached the rank of Assistant Engineer, when he was compelled to give up active work on account of ill health.

Mr. Wachtel was extremely well liked by all his associates, and it was with great sorrow that his many friends learned of his death on October 10th, 1917, at Welles, N. Y., where he had spent the last few years of his life. By his work and conduct, which were always beyond criticism, he gave promise of going far in his profession.

He was married in 1913, to Miss Pauline L. Cohn, of Saratoga Springs, N. Y., who survives him.

Mr. Wachtel was elected a Junior of the American Society of Civil Engineers on March 2d, 1909, and an Associate Member on September 2d, 1914.

* Memoir prepared by J. H. Sturdevant, Assoc. M. Am. Soc. C. E.

ROGER TAPPAN, Jun. Am. Soc. C. E.*

Died December 6th, 1916.

Roger Tappan, the son of the Rev. Daniel Dana Tappan and Abigail (Marsh) Tappan, was born in Marshfield, Mass., on November 27th, 1848.

After his graduation from the Massachusetts Institute of Technology, Mr. Tappan was employed as an Assistant Engineer on the northern extension of the old Eastern Railroad. He was afterward engaged as an Assistant on the Engineering Staff of the Fitchburg Railroad during the construction of the Hoosac Tunnel, and had charge of railroad surveys in the Adirondack Mountains, as well as in several of the Western States. His last engineering work was as Engineer in charge of the survey for the North Conway and Mt. Kearsarge Railroad, a proposed line up Mt. Kearsarge, in New Hampshire, which was never constructed.

As a young man, Mr. Tappan had taught drawing in the public schools of Salem and Haverhill, Mass., and about 1890, he gave up engineering work to devote his time to painting in water colors and oils. He studied art in Boston, New York City, and Paris, and spent much of the latter part of his life abroad.

He died on December 6th, 1916, at Natick, Mass., and was buried at Topsfield, Mass. He is survived by his second wife, who was Miss Anna Moberg, of Sjundea, Finland, his first wife, Miss Elizabeth Carleton, of Haverhill, Mass., having died in 1890.

Mr. Tappan was elected a Junior of the American Society of Civil Engineers on December 3d, 1884.

* Memoir prepared by the Secretary from information on file at the Headquarters of the Society.

NELSON JAMES WELTON, F. Am. Soc. C. E.*

DIED JUNE 5TH, 1917.

Nelson James Welton was born on February 15th, 1829, in the Bucks Hill District of Waterbury, Conn., in the house which was held, by inheritance, by six generations of descendants of Richard Welton who is said to have been the first male child born in Waterbury.

Mr. Welton received his education in the district schools and the old Waterbury Academy, where he studied surveying and civil engineering. In 1850, with his appointment as County Surveyor of New Haven County, he began a career as a civil engineer which lasted for more than half a century.

From 1853 to 1885, he served as City Surveyor of Waterbury; he also had charge of the construction of that city's water-works, and was its General Manager for more than 30 years, his record for economy of operation being exceptional. When the city's first sewerage system was ordered to be built, Mr. Welton was employed as Engineer in charge of its construction. In 1878, he was appointed a member of the Connecticut State Board of Civil Engineers, which Board was organized to inspect the reservoirs and dams in the State, and continued as a member until he resigned in 1902.

Aside from his engineering activities, Mr. Welton held many important public offices, serving as City Clerk from 1853 to 1858; Town Clerk from 1856 to 1863; and for twenty-eight years he was a Justice of the Peace. In 1859, he was elected a Judge of the Probate Court; in 1861, as Representative from Waterbury to the State General Assembly; in 1863-64, as Recorder of the City Court; and from 1867 to 1896 (with the exception of two years) he was President of the Board of Water Commissioners. In 1853, when the Riverside Cemetery Association was organized, he was made Superintendent, and had also served as its Treasurer since 1865.

Mr. Welton was greatly interested in many of Waterbury's public institutions, and rendered valuable service in securing the bequest for, and the establishment of, the Bronson Public Library in 1870. He was an Incorporator and a Director of the Waterbury Savings Bank, a Director of the Waterbury National Bank, and a member of the Corporation of and, for a time, Treasurer of St. Margaret's School for Girls.

He was a life long member of St. John's Protestant Episcopal Church, having been connected with its Sunday School for 52 years

* Memoir prepared by the Secretary from information furnished by William G. Smith, Esq., Waterbury, Conn.

and having served as its Senior Warden from 1889 until his death. Mr. Welton was a Mason, a Knight Templar, and a Shriner.

He was a man of quiet nature, and fond of home life. He was also a good business man, one of his most marked characteristics being his sound, level-headed judgment. Having been widely known in Waterbury through his church, fraternity, and business associations, he was held in great respect and deep affection by those who knew him.

Mr. Welton was a Charter Member of the Connecticut Society of Civil Engineers, and was elected a Fellow of the American Society of Civil Engineers on January 20th, 1873.

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